LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
CONFLUENCE SITE (7B-1)

FEASIBILITY DESIGN REPORT

September, 1985

Prepared For:

Alberta Environment
Planning Division

Prepared By:
W-E-R Engineering Ltd.
in association with
Thurber Consultants Ltd.
and
Nanuk Engineering and Development Ltd.

September 30, 1985

Alberta Environment
Planning Division
200 - 5 Avenue South
Lethbridge, Alberta
T1J 4C7

Attention: Mr. Sandy Larsen, P.Eng.

Re: Little Bow River Basin
Storage Investigation, Confluence Site (7B-1)
Feasibility Design Report

Dear Mr. Larsen:

W-E-R Engineering Ltd. is pleased to submit herewith its final report entitled "Little Bow River Basin Storage Site Investigation Confluence Site (7B-1) - Feasibility Design Report", September, 1985.

The investigation was conducted in association with Thurber Consultants Ltd. who were responsible for the geotechnical assessment, and Nanuk Engineering and Development Ltd. who carried out the water supply and hydrology studies.

This report outlines the results of feasibility design studies undertaken for three reservoir storage alternatives of 30, 50 and 70 thousand acre-feet at Little Bow Site 7B-1.

Our firm appreciates the opportunity to be involved in the development of storage in this water-short area of Southern Alberta. It has been a most challenging assignment.

We would be pleased to meet with you at your convenience to discuss the results of this investigation.

Yours very truly,

W-E-R ENGINEERING LTD.

J. M. Wise, P.Eng.
Vice-President

JMW:pc

PROFESSIONAL ENGINEER ALBERTA J. M. WISE

THE ASSOCIATION OF PROFESSIONAL ENGINEERS GEOLOGISTS and GEOPHYSICISTS OF ALBERTA
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SYNOPSIS

Conceptual design studies conducted by W-E-R Engineering Ltd. examined three storage sites below the confluence of the Little Bow River and Mosquito Creek - Little Bow Sites 7A, 7B and 7B-1, and recommended that Site 7B-1 be retained for more detailed investigation.

This report describes the engineering studies carried out by W-E-R, which confirm the technical feasibility of three storage alternatives of the selected reservoir site, 7B-1, and provides hydrology and water supply, geotechnical, feasibility design and reservoir damage investigations for the storage under consideration.

The total estimated costs for the three reservoir storage alternatives are as follows:

Storage Capacity	Project Cost	Cost per Ac.Ft.* of Basin Yield
30,000 ac.ft.	$ 9,806,000	$158
50,000 ac.ft.	$11,308,000	$149
70,000 ac.ft.	$13,587,000	$166

The report recommends that a reservoir storage capacity in the range of 50,000 to 70,000 acre-feet be considered for implementation.

*Based on new 60 percent guideline with 60 cfs (Squaw Coulee) and 300 cfs (Little Bow) diversion capacities. Does not include cost of increasing diversion capability.

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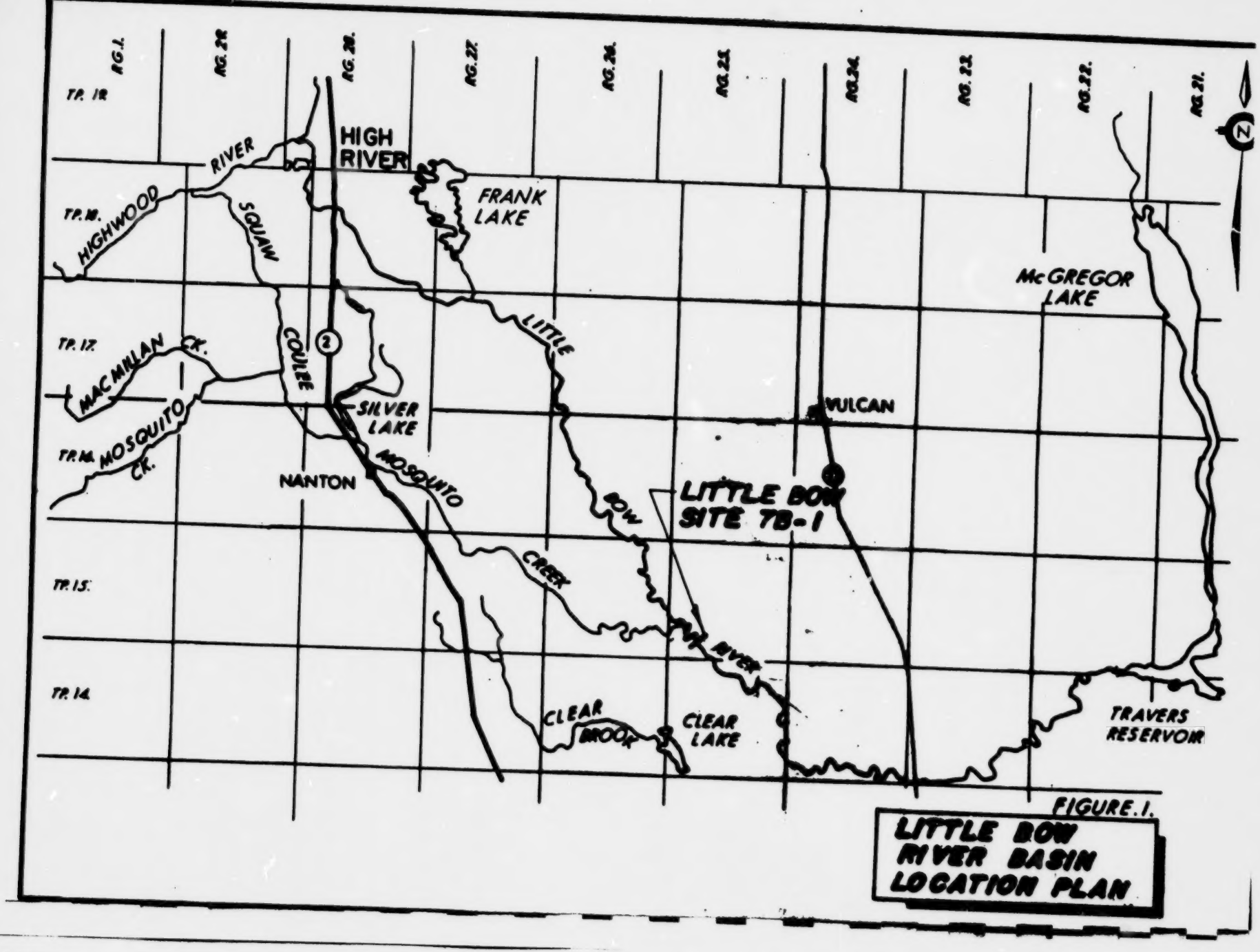


FIGURE.1.

LITTLE BOW RIVER BASIN LOCATION PLAN

1.0 INTRODUCTION

Irrigation water shortages within the Little Bow River Basin in recent years resulting from a combination of drought coupled with high demand have clearly demonstrated the need for storage development within the basin.

Alberta Environment's on-going Phase II studies which constitute a water management plan for the basin have included a requirement for storage reservoir investigations. Accordingly, W-E-R Engineering Ltd. has been retained to conduct a number of damsite investigations related to storage within the Little Bow Basin.

Current studies by W-E-R Engineering have encompassed storage reservoirs immediately below the confluence of the Little Bow River and Mosquito Creek. The recently completed conceptual study (Reference 2) examined three confluence sites and recommended that Little Bow Site 7B-1 be retained for feasibility studies.

The present report describes the results of a feasibility level evaluation of Site 7B-1 (see Figure 1). Three reservoir storage alternatives of 37 000, 61 700 and 86 300 dam^3 (30,000, 50,000 and 70,000 ac.ft.) have been considered.

The feasibility of the three reservoir options as a potential source of supply for highline irrigation canals has also been considered in this investigation.

Confluence site investigations have also incorporated the new 60 percent (of Highwood River flows) guideline, with diversion rates for Squaw Coulee set at the present system capacity of 1.7 m^3/s (60 cfs), and capacities at the Little Bow Canal ranging from 2.8 m^3/s (100 cfs) to 14.2 m^3/s (500 cfs).

This report compares each of the three reservoir alternatives at Site 7B-1 on the basis of water supply capability, geotechnical considerations, structure layout and design, reservoir damages, land acquisition, and costs of implementation. The study results have also been summarized in the form of design layouts, pertinent data, and cost summaries at the back of the main report.

Details of each of the study components are included in the following appendices:

- o Appendix A - Water Supply and Hydrology
- o Appendix B - Geotechnical
- o Appendix C - Design and Costing
- o Appendix D - Reservoir Costs

On the basis of the feasibility design investigations, recommendations for implementation are presented.

2.0 HYDROLOGIC ASSESSMENT

Water supply and flood flow assessments were made at a feasibility level for the confluence site, 78-1. The results were used to support selection of reservoir storage capacity and spillway design. Details of this hydrologic assessment are provided in Appendix A.

2.1 Water Supply

The water supply capability of site 78-1 was evaluated by determining the total basin yield provided by three levels of storage - 36 900, 61 500 and 86 100 dam^3 (30,000, 50,000 and 70,000 ac.ft.) Basin yield is the total water available to meet all requirements, but primarily for irrigation use, and is not synonymous with reservoir release. The capacity of the Little Bow Canal was also varied from 2.83 to 14.2 m^3/s in steps of 2.83 m^3/s to assess the effect of increased diversion capability from the Highwood River.

a) Data Base

Water available from the Highwood River for diversion to the Little Bow Basin was based on the following operational guideline;

- o maximum volume diverted cannot exceed 60% of the daily flow in the Highwood at High River,
- o a residual flow of 1.98 m^3/s (70 cfs) must remain in the Highwood downstream of the headgate for the Little Bow Canal.

The Squaw Coulee diversion capacity was set at 60 cfs for all evaluations. The operating season for the two diversion canals was assumed to be April 15 to October 15.

The Squaw Coulee diversion capacity was set at 1.70 m^3/s (60 cfs) for all evaluations. The operating season for the two diversion canals was assumed to be April 15 to October 15.

b) Methodology and Results

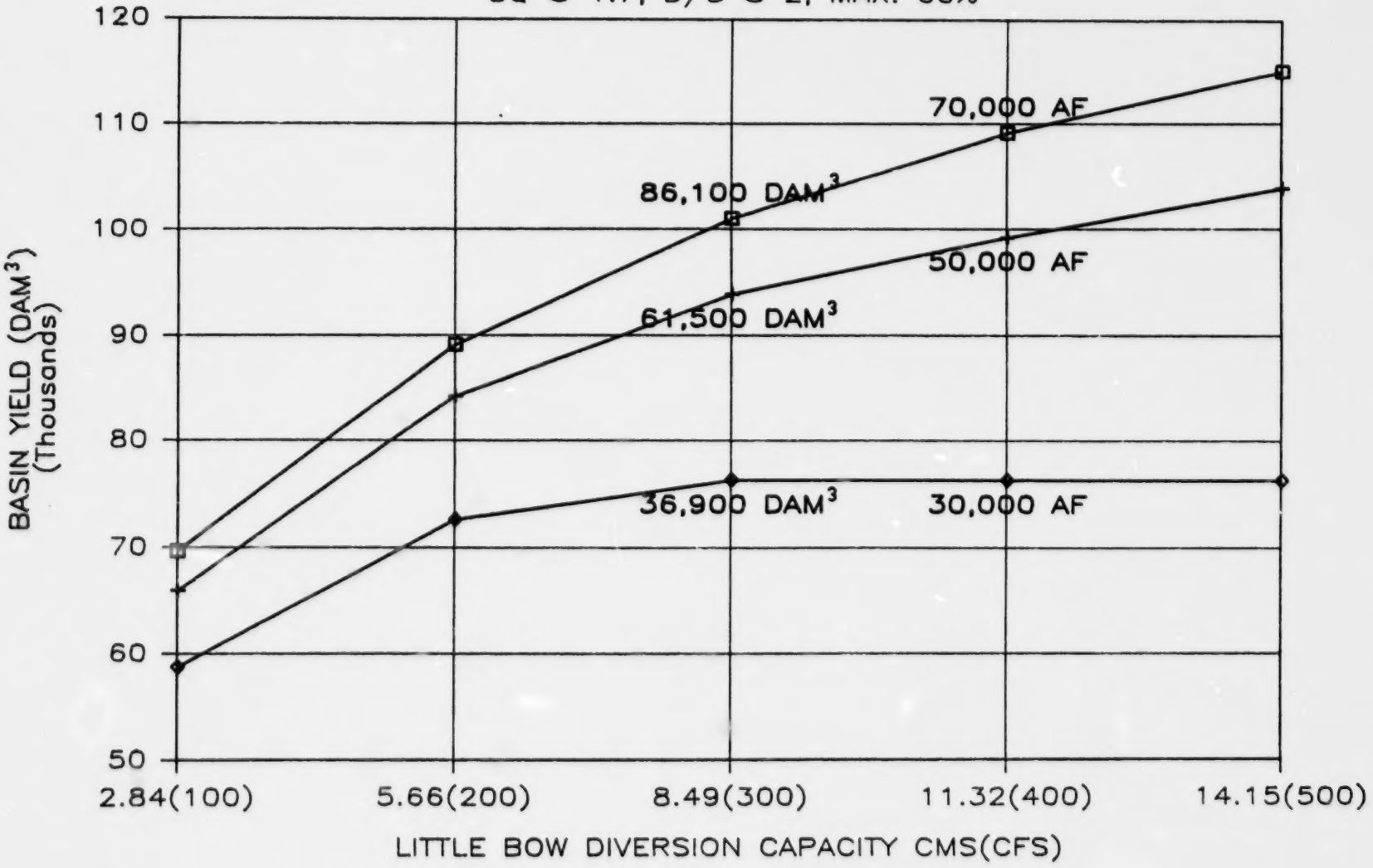
The period 1916 to 1941 was selected for the water supply study since it included the most critical drought period on record. Water in excess of the total basin requirement, in a particular month, was stored in the reservoir. Stored water at the end of the irrigation season was carried forward into the next year. The basin requirement was constant throughout one simulation. The water requirement was incremented, in subsequent simulations, until a deficit occurred. This requirement then became the total basin yield for the simulated reservoir storage. Diversion capacity was then changed and a new basin yield determined by iteration. The water balance model considered all inflows (diversions and natural), evaporation, but not channel losses.

Based on these studies, a capacity for the Little Bow Canal can be selected and an appreciation of the effectiveness of increasing storage can be gained. Figure 2 illustrates the effect of increasing diversion canal capacity and changing the storage capacity of the reservoir. The following observations can be made from Figure 2:

1. A canal capacity between 5.67 (200 cfs) and 8.50 (300 cfs) m^3/s produces appreciable increases in basin yield over the 2.83 m^3/s (100 cfs) capacity. Beyond 8.50 m^3/s (300 cfs), yield increases are relatively small.

LITTLE BOW CONFLUENCE SITE

SQ @ 1.7, D/S @ 2, MAX. 60%



W-E-R ENGINEERING LTD.

NANUK Engineering
Figure 2

2. The 61 500 dam^3 (50,000 ac.ft.) reservoir produces a yield similar to the 86 100 dam^3 (70,000 ac.ft.)reservoir at 8.50 m^3/s (300 cfs) diversion capacity for both. The 24 600 dam^3 (20,000 ac.ft.) increase in storage produces a 7 400 dam^3 (6,000 ac.ft.) change in yield.

2.2 Flood Flows

Flood frequency analyses were carried out for use in the design of spillways and construction diversion works for each site. The probable maximum flood (PMF) was determined using Saskatchewan-Nelson Basin Board data. The 1 in 100 year and 1 in 25 year floods were determined using frequency analysis from recorded maximum daily flows of the Little Bow River at Carmangay. A complete description of the methodology can be found in Appendix A.

The flood flows for preliminary design are:

PMF - 570 to 700 m^3/s (20,000 to 25,000 cfs) (maximum mean daily flow)

1:25 year - 85 m^3/s (3,000 cfs) (peak instantaneous)

1:100 year - 150 m^3/s (5,200 cfs) (peak instantaneous)

3.0 GEOLOGICAL/GEOTECHNICAL

This section presents a preliminary geotechnical evaluation of a damsite located on the Little Bow River about 20 km southeast of Parkland, Alberta. The site has been designated as Site 78-1. Three different reservoir storage capacities are being considered at this site, and therefore, three different embankment heights have been examined.

Details of the geotechnical evaluation are presented in Appendix B.

3.1 Field Work

A comprehensive drilling program was carried out by PFRA in 1971 (Reference 1) at the currently proposed damsite for the 86 300 dam^3 (70,000 ac.ft.) reservoir. For the 37 000 and 61 700 dam^3 (30,000 and 50,000 ac.ft.) alternatives the dam centreline has been relocated about 200 m upstream from this centreline as described in Section 4.3. Subsurface information from the original site has therefore been extrapolated upstream for the preliminary geotechnical evaluation of the smaller storages. If a decision is made to proceed with construction, a comprehensive drilling program should be carried out along the centreline finally selected.

3.2 Site Geology

The stratigraphy at the proposed damsite is relatively complex. At this location, the Little Bow River has cut down through about 20 m of glacial clay till and about 10 m into the underlying shale and sandstone bedrock. An alluvial terrace is present on the west (right) valley wall and

the clay till in this area is overlain by 3 to 6 m of granular alluvium. Alluvial materials also overlie the clay till on the uplands to the east of the valley. This reach of the Little Bow River follows the preglacial Tee Pee Valley. As a result, at the proposed dam site, granular preglacial alluvium is present, sandwiched between the clay till and the bedrock surface.

3.3 Reservoir Shoreline Stability

Three deep-seated landslides have been noted along the valley walls upstream from the dam embankment. Detailed geotechnical investigations will be required to assess the effects of reservoir impoundment on the stability of these landslides.

No significant concerns with respect to reservoir shoreline stability were identified in any other locations within the reservoir. Deep-seated landslides are possible in some areas, however, the frequency of occurrence of such landslides is expected to be very low. Shallow instability can be expected along the reservoir shorelines due to wave erosion. The beaches are expected to be self stabilizing over 20 to 30 years.

3.4 Reservoir Watertightness

Lateral seepage from the reservoir is not a major concern at this time. A detailed hydrogeological investigation will be required during detailed design in order to confirm this assessment.

3.5 Embankment Cross-section

A zoned earthfill embankment has been recommended for all three dam embankment heights. It is recommended that the downstream slopes are constructed at an average 3 horizontal to 1 vertical, while the upstream slopes would be constructed at an average 4 horizontal to 1 vertical. It is anticipated that the slopes can be bermed in order to reduce the quantity of fill required for the embankments. A decision in this regard will depend on the results of material balance studies, laboratory test results and stability analyses which should be undertaken during detailed design.

The failure mechanism of the three landslides located upstream from the proposed dam site should be established. The results of that work will have an impact on the soil parameters used for the design of the dam embankment. It is possible that the embankment slideslopes may have to be flattened somewhat from those recommended here.

3.6 Seepage Below Embankment

A conventional downstream drainage blanket and toe drain should be incorporated into the embankment design. Relief wells may be required at the downstream toe, however, more detailed subsurface information is required before a decision can be made in ths regard. A compacted clay cut off must be used below the centreline of the embankment in order to prevent seepage through the alluvial sediments below the embankment. The cutoff must be excavated through 7 to 10 m of alluvium.

3.7 Abutment Seepage

Seepage around the abutments, through fractures and joints in the bedrock, is a concern at this site. Preparation of the abutments, as outlined in the report, will be required. It may be necessary to grout the rock at the abutments, however, further field investigations should be undertaken before a decision is made in this regard.

3.8 Spillways

A single spillway, excavated around the east (left) dam abutment is being proposed at this site. The depth of excavation for the spillway will depend on the maximum reservoir operating level ultimately selected. The excavation will be carried out primarily through clay till and, in the case of the smallest reservoir, into the underlying bedrock. No significant construction difficulties with any of these excavations are anticipated. Blasting of cemented sandstone layers will be required if such material is encountered within the bedrock. Permanent slopes in clay till should be cut at 3 horizontal to 1 vertical. Permanent slopes in the bedrock should be cut at 2 horizontal to 1 vertical, primarily to minimize surface erosion of the shale. If steeper slopes are desired in the bedrock, surface treatment, such as shotcrete, should be considered.

3.9 Low Level Conduits

The low level conduit should be founded either on bedrock, or on compacted silty clay fill which extends below the conduit down to the bedrock. The conduit should therefore be located where the depth to bedrock is a minimum.

3.10 Concrete

Sources for concrete aggregate have not been confirmed. It is anticipated that suitable aggregate (which will have to be crushed, screened and washed) can be located in the alluvial sediments upstream from the dam site. It is recommended that Type 50 sulphate resistant cement be used for all concrete in contact with soil at this site.

3.11 Additional Geotechnical Investigations

Prior to final design, a comprehensive drilling program will be required in order to confirm subsurface conditions at the proposed site.

Other geotechnical studies, as detailed in the report, will also be required.

It is recommended that some phases of these geotechnical investigations be carried out at the pre-design stage.

4.0 DESIGN STUDIES

4.1 General

Feasibility design studies were carried out for each of the three reservoir alternatives under consideration at Little Bow Site 7B-1. These investigations covered structure layouts, hydraulic and structural designs and costing for the main dam and appurtenant works. An assessment of the reservoirs to supply highline irrigation canals has also been considered. Details of the design studies are presented in Appendix C.

The feasibility studies utilized the current water supply, hydrology, and geotechnical evaluation in support of designs for the main dam, spillway and conduit structures.

Topographic mapping produced for the conceptual design study (Reference 2) at a scale of 1:10,000 with 2 metre contour intervals was again utilized to generate area capacity curves and determine damage costs for the reservoir alternatives.

4.2 Project Requirements

Basic project requirements of the present study included: a) the evaluation of three alternative reservoir sizes for Site 7B-1 - 37 000, 61 700 and 86 300 dam^3 (30,000, 50,000 and 70,000 ac.ft.); b) controlling flood events set at 1:100 for the spillway structure design flood and 0.5 PMF for the project design flood with appropriate freeboard allowances for wind setup, wave runup, gate operation failure and frost action; c) downstream irrigation requirements established at 8.5 m^3/s (300 cfs); and d) an examination of the potential for highline irrigation canals originating from each of the three storage alternatives.

4.3 General Layout

General arrangements of the dam and appurtenant structures are shown in Figures 3, 4 and 5 with pertinent data provided in Tables 1, 2 and 3.

For the 86 300 dam^3 (70,000 ac.ft.) reservoir alternative the dam centreline location proposed by PFRA in 1971 (Reference 1) was retained. However, the embankment centrelines for the 61 700 dam^3 (50,000 ac.ft.) and 37 000 dam^3 (30,000 ac.ft.) alternatives were located approximately 200 m upstream of the PFRA site as in the case of the conceptual design study (Reference 2). This was done so that the east gully could form part of the spillway approach channel to reduce excavation quantities. Embankment designs are based on the geotechnical investigations presented in this report.

4.4 Spillways

Reinforced concrete, gated chute spillways with hydraulic jump stilling basins are proposed for handling floods in each alternative. The spillway structures would be equipped with automatically operated, motorized radial gates.

Spillway crest length and gate height combinations have been optimized to provide the most cost effective arrangement. Chute structures have been designed with a flared configuration to ensure that flow energy is adequately dissipated.

The selection of spillway structural components (sizing of wall and slab thicknesses) has been made on the basis of

existing structures of a similar type. Conventional cut-offs, key walls, joints and drainage systems have been incorporated into the design to accommodate movements and control foundation seepage.

4.5 Irrigation Conduits

Irrigation conduits are essentially low level outlet structures designed to meet the downstream irrigation demand. The outlet conduits, as proposed, consist of precast concrete pipelines with reinforced concrete inlet, gatewell and outlet structures. The gatewells would be two-celled structures equipped with both service and auxiliary (emergency) gates for the control of irrigation flows. The conduits will also serve as temporary diversion works during the construction stage of the project.

4.6 Highline Canals

Preliminary layouts of highline irrigation canals are proposed for each of the three alternative storages under consideration. These canal layouts are based on the assumption that 50 percent of the storage capacity of each of the reservoir alternatives will be available for irrigation supply to the respective highline canals, with the other 50 percent of storage used for lowline irrigation. Numerous other storage combinations would of course be considered in the final design stage, however, this was not a part of the scope of the current investigation. Neither irrigation headworks nor canals were costed for this study.

4.7 Construction Cost Estimates

Cost estimates for the main dam and appurtenant structures, including allowances for contingencies and engineering, are provided in Tables 1, 2 and 3. Unit prices used in preparing these estimates are based on recent bid prices for similar projects in Southern Alberta.

4.8 Project Implementation

It is estimated that final designs and geotechnical studies and the preparation of tender documents can be completed in about four months. Construction of any of the three damsites under consideration will require two seasons. Ideally, designs could be carried out over winter with tenders called the following spring. Winter construction of structural components would also be feasible.

5.0 RESERVOIR COSTS

An estimate of the costs of land acquisition and reservoir damages has been prepared for each of the three reservoir capacities, as summarized in the following sections of this report. Details of the reservoir costs are included in Appendix D.

5.1 Basis of Costing

Reservoir land costs are based on the total land control requirements taken to the crest of the dam and include an allowance for severance and acquisition.

Reservoir damages include the cost of roads and bridges, power lines, gas pipeline, water intake system, AGT lines and relocation of farm buildings.

Total reservoir costs for each site are provided in Tables 1, 2 and 3.

5.2 Little Bow Site 78-1 - 37 000 dam^3 (30,000 ac.ft.)

a) Land Acquisition

A total of 776 ha of land to the top-of-dam elevation (964.1 m) are affected, 176 ha of which are cultivated.

b) Roads and Bridges

Three alternate routes were considered for secondary Highway 529: a causeway, a north relocation and a south relocation. The causeway was not deemed to be practical. The most economic option is a 3.2 km re-routing of the road to the south.

c) **Farm Buildings**

Three farmsteads and the Little Bow River Hutterite Colony will require relocation.

d) **Utilities**

A TransAlta single phase line crosses the reservoir in the SE 26-15-26-W4 serving two customers in the SE-26-15-W4 and the NW-24-15-26-W4. A new service to the NW-24-15-26-W4 could be provided from the east with construction of 2 km of new line.

No pipelines were found to be affected by the reservoir with the exception of natural gas service lines to farm buildings which are to be relocated.

The Town of Vulcan water intake and pumphouse in the NE 35-15-26-W4 would have to be moved northeast to above top-of-dam elevation. This would require removal of some existing supply line and construction of a new intake line and pumphouse.

One short section of AGT telephone line in the SW 18-15-25 would have to be relocated.

5.3 Little Bow Site 78-1 - 67 100 dam^3 (50,000 ac.ft.)

a) **Land Acquisition**

A total of 1,139 ha of land to the top-of-dam elevation (967.7 m) are affected, 327 ha of which are cultivated.

b) Roads and Bridges

Three possible routes were evaluated for secondary Highway 529 as indicated in Section 5.2 b), with the most economical option being a 3.2 km re-routing of the road to the south.

c) Farm Buildings

Three farmsteads and the Little Bow River Hutterite Colony will require relocation.

d) Utilities

Utility relocations for this alternative remain the same as outlined in Section 5.2 (d) of this report.

5.4 Little Bow Site 7B-1 - 86 300 dam^3 (70,000 ac.ft.)

a) Land Acquisition

A total of 1,783 ha of land to the top-of-dam elevation (970.3 m) are affected, 353 ha of which are cultivated as hayland or crop land.

b) Roads and Bridges

i) Three possible routes were evaluated for secondary Highway 529 as indicated in Section 5.2 b), with the most economical option being a 3.2 km re-routing of the road to the south.

ii) The existing timber farm bridge in NW 9-16-26-W4 would be abandoned and local traffic re-routed over secondary Highway 533.

c) **Farm Buildings**

Four farmsteads and the Little Bow River Hutterite Colony will require relocation.

d) **Utilities**

Utility relocations remain as outlined in Sections 5.2 (d) of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1. It is technically feasible to construct a storage reservoir at Little Bow Site 7B-1 for any of the three capacities under consideration.

2. Highline irrigation canals from Site 7B-1 likewise appear to be technically feasible.

3. An optimized location of the final dam centreline is still required to ensure minimum contact with the preglacial Tee Pee Valley.

4. The total estimated costs for the three storage alternatives are as follows:

Site 7B-1 Storage dam^3 (ac.ft.)	Project Cost	Cost/dam^3 (ac.ft.)* of Yield
37 000 (30,000)	$ 9,806,000	$128 (158)
61 700 (50,000)	$11,308,000	$120 (149)
86 300 (70,000)	$13,587,000	$134 (166)

*Based on new 60 percent guideline with 60 cfs (Squaw Coulee) and 300 cfs (Little Bow) diversion capacities. Does not include cost of increasing diversion capability.

5. The total project costs for the 61 700 dam^3 (50,000 ac.ft.) storage level have not changed appreciably from the conceptual study. However, construction costs have decreased by about 9 percent while reservoir damages have increased approximately 15 percent. The lower

construction costs are attributable to optimized design in the feasibility stage. The increased damages are due primarily to the identification of additional land acquisition in the upper reservoir and the cost of the Vulcan water intake.

6. Based on the cost per dam^3 (acre-foot) of basin yield the full range of storage potential as investigated in this study is considered economical.

6.2 Recommendations

Based on the feasibility design studies conducted for the three storage levels and the conclusions reached, it is recommended that:

1. Consideration be given to the implementation of Little Bow Site 78-1 in the storage range of 61 700 to 86 300 dam^3 (50,000 to 70,000 ac.ft.).

2. In the final design stage a comprehensive drilling program be conducted along the selected dam centreline to confirm subsurface conditions at the proposed site.

3. Consideration be given to a pre-design stage investigation, including a proportional drilling program, to finalize the dam centreline prior to implementation of the project.

4. Such a pre-design study could also incorporate a more detailed examination of the highline supply canals.

LIST OF REFERENCES

1. "Bow River Diversion to Oldman River Appendix B Diversion Works - April, 1971" by PFRA.

2. "Little Bow River Basin Storage Site Investigation, Confluence Sites, Conceptual Design Report - August 1985" by W-E-R Engineering Ltd.

TABLE 1
Pertinent Data and Cost Summary
Little Bow Site 78-1
37 000 dam^3 (30,000 ac.ft.) Storage Alternative

Reservoir

Full Supply Level (FSL)	961.2 m
Total Storge at FSL	37 000 dam^3
Flooded Area at FSL	575 ha
Project Design Flood - 0.5 PMF (inflow)	322.8 m^3/s
Spillway Design Flood - 1:100 (inflow)	147.3 m^3/s

Embankment

Top of Dam Elevation	964.1 m
Maximum Height	19.0 m
Volume of Fill	394 600 m^3
Freeboard at FSL	2.9 m

Spillway

Crest Elevation	959.1 m
Radial Gates	3 @ 4.0 m (W) x 2.1 m (H)
Basin Elevation	943.3 m
Design Discharge (1:100 routed outflow)	99.6 m^3/s
Probable Maximum Discharge (0.5 PMF routed outflow)	265.0 m^3/s

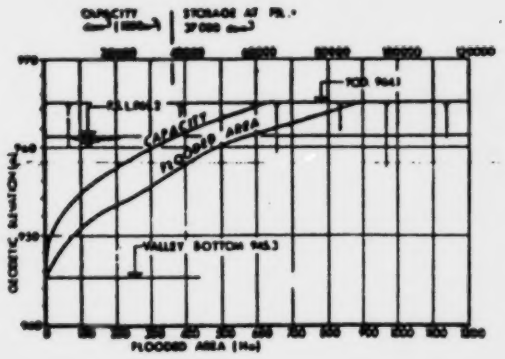
Irrigation Supply Conduit

Conduit Diameter	1.5 m
Irrigation Supply Capacity Requirement	8.5 m^3/s
Gate Size	2 @ 2 m x 2 m
Intake Crest Elevation	949.0 m
Minimum Reservoir Elevation to Pass Irrigation Requirement	949.7 m

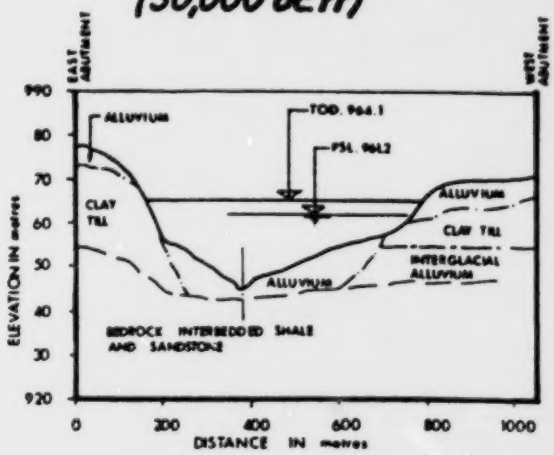
Cost Estimate Summary

Care of Water	210,000
Embankment	2,023,000
Spillway	1,632,000
Irrigation Supply Conduit	541,000
Subtotal	$4,406,000
Contingencies (20%)	881,000
Subtotal	5,288,000
Engineering (15%)	793,000
Reservoir Costs	3,725,000
Total Project Cost	$9,806,000

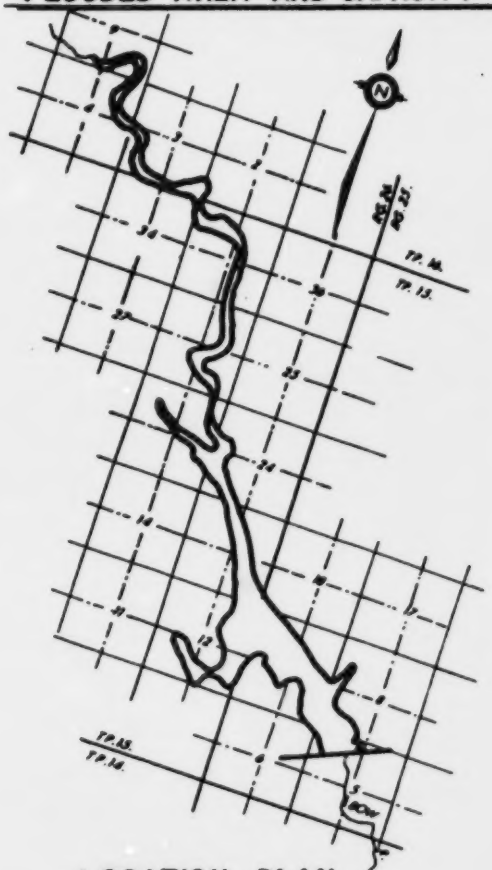
LITTLE BOW SITE 7B-1 37,000 dam³ (30,000 ac.ft)



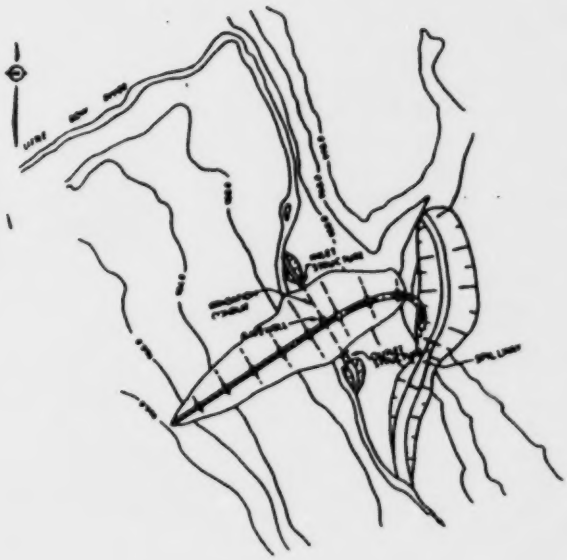
FLOODED AREA AND CAPACITY CURVES



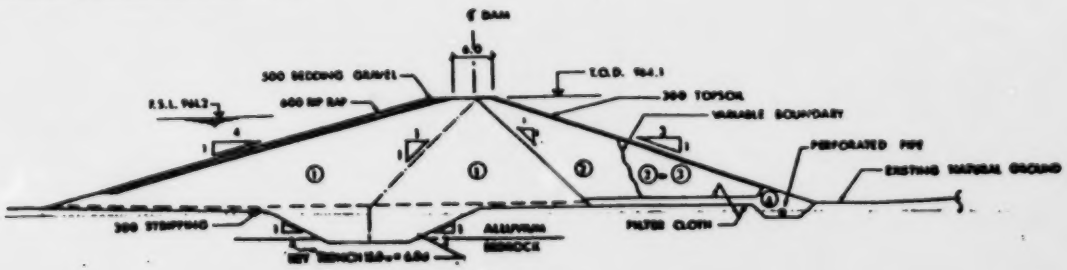
VALLEY CROSS SECTION AT DAMSITE



LOCATION PLAN



SITE LAYOUT



MAXIMUM EMBANKMENT SECTION

FIGURE. 3

TABLE 2
Pertinent Data and Cost Summary
Little Bow Site 7B-1
61 700 dam^3 (50,000 ac.ft.) Storage Alternative

Reservoir

Full Supply Level (FSL)	964.7 m
Total Storage at FSL	61 700 dam^3
Flooded Area at FSL	865 ha
Project Design Flood - 0.5 PMF (inflow)	322.8 m^3/s
Spillway Design Flood - 1:100 (inflow)	147.3 m^3/s
Freeboard at FSL	3.0 m

Embankment

Top of Dam Elevation	967.4 m
Maximum Height	22.7 m
Volume of Fill	574 000 m^3

Spillway

Crest Elevation	962.6 m
Radial Gates	3 @ 3.5 m (W) x 2.1 m (H)
Basin Elevation	943.1 m
Design Discharge (1:100 routed outflow)	84.6 m^3/s
Probable Maximum Discharge (0.5 PMF routed outflow)	239.2 m^3/s

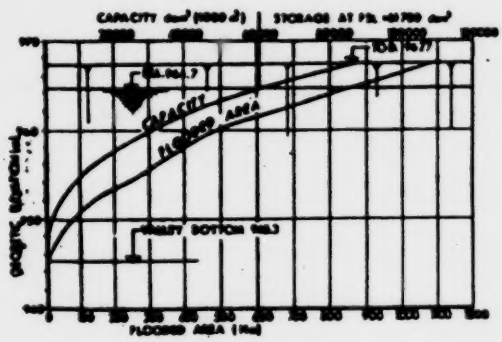
Irrigation Supply Conduit

Conduit Diameter	1.5 m
Irrigation Supply Capacity Requirement	8.5 m^3/s
Gate Size	2 @ 2 m x 2 m
Intake Crest Elevation	949.0 m
Minimum Reservoir Elevation to Pass Irrigation Requirement	949.7 m

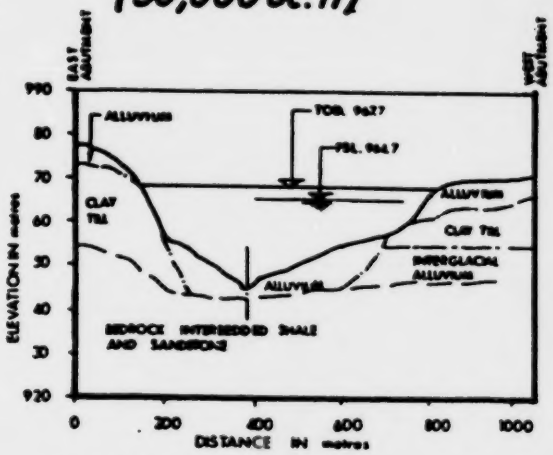
Cost Estimate Summary

Care of Water	242,000
Embankment	2,783,000
Spillway	1,429,000
Irrigation Supply Conduit	628,000
Subtotal	$5,082,000
Contingencies (20%)	1,016,000
Subtotal	6,098,000
Engineering (15%)	915,000
Reservoir Costs	4,295,000
Total Project Cost	$11,308,000

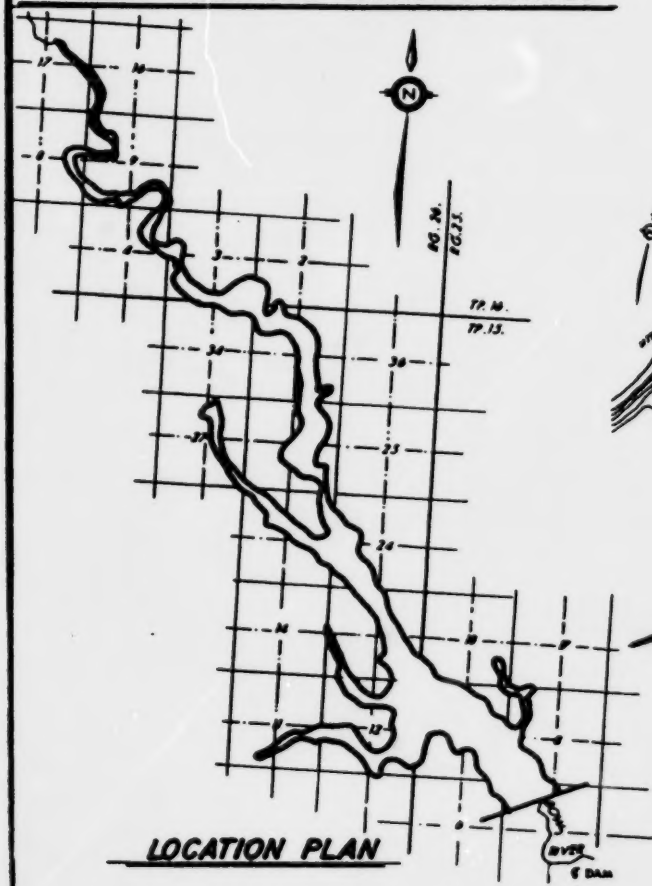
LITTLE BOW SITE 7B-1 61,700 dam³ (50,000 ac. ft)



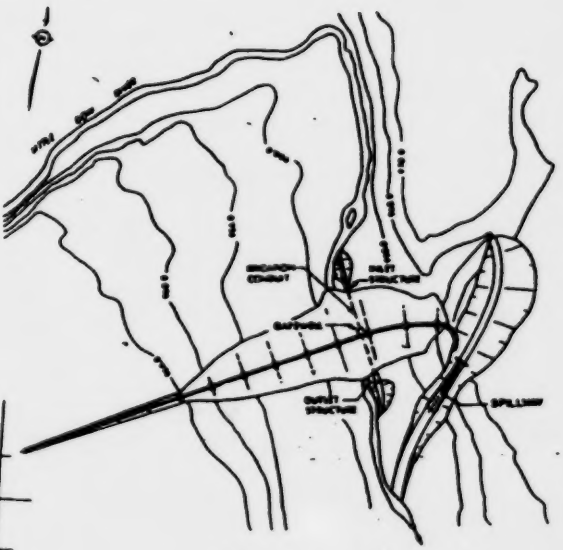
FLOODED AREA AND CAPACITY CURVES



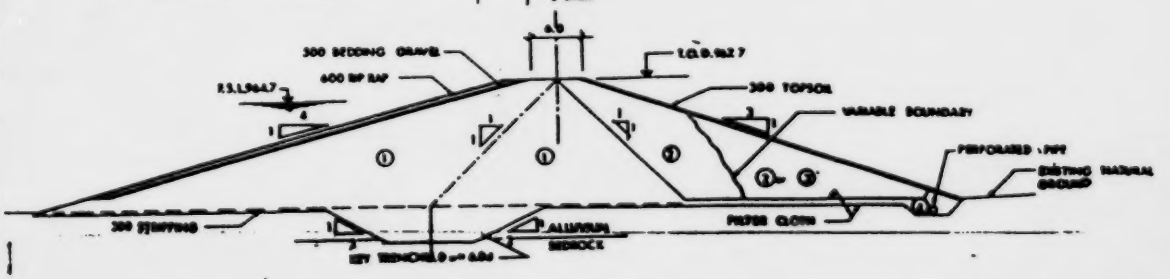
VALLEY CROSS SECTION AT DAMSITE



LOCATION PLAN



SITE LAYOUT



MAXIMUM EMBANKMENT SECTION

FIGURE . 4.

TABLE 3
Pertinent Data and Cost Summary
Little Bow Site 7B-1
86 300 dam^3 (70,000 ac.ft.) Storage Alternative

Reservoir

Full Supply Level (FSL)	967.2 m
Total Storage at FSL	86 300 dam^3
Flooded Area at FSL	1,075 ha
Project Design Flood - 0.5 PMF (inflow)	322.8 m^3/s
Spillway Design Flood - 1:100 (inflow)	147.3 m^3/s
Freeboard at FSL	3.1 m

Embankment

Top of Dam Elevation	970.3 m
Maximum Height	25.0 m
Volume of Fill	800 000 m^3

Spillway

Crest Elevation	965.1 m
Radial Gates	3 @ 3.3 m (W) x 2.1 m (H)
Basin Elevation	943.0 m
Design Discharge (1:100 routed outflow)	78.6 m^3/s
Probable Maximum Discharge (0.5 PMF routed outflow)	228.9 m^3/s

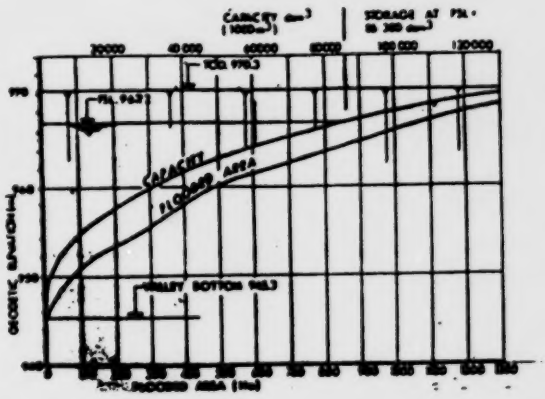
Irrigation Supply Conduit

Conduit Diameter	1.5 m
Irrigation Supply Capacity Requirement	8.5 m^3/s
Gate Size	2 @ 2 m x 2 m
Intake Crest Elevation	949.0 m
Minimum Reservoir Elevation to Pass Irrigation Requirement	949.7 m

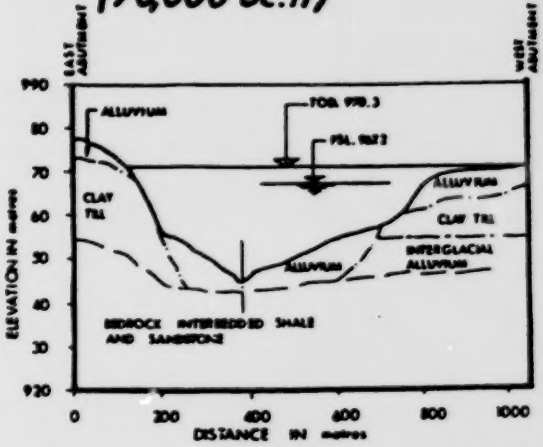
Cost Estimate Summary

Care of Water	295,000
Embankment	3,876,000
Spillway	1,335,000
Irrigation Supply Conduit	694,000
Subtotal	$6,200,000
Contingencies (20%)	1,240,000
Subtotal	7,440,000
Engineering (15%)	1,116,000
Reservoir Costs	5,031,000
Total Project Cost	$13,587,000

LITTLE BOW SITE 7B-1 86,300 dam³ (70,000 ac.ft)

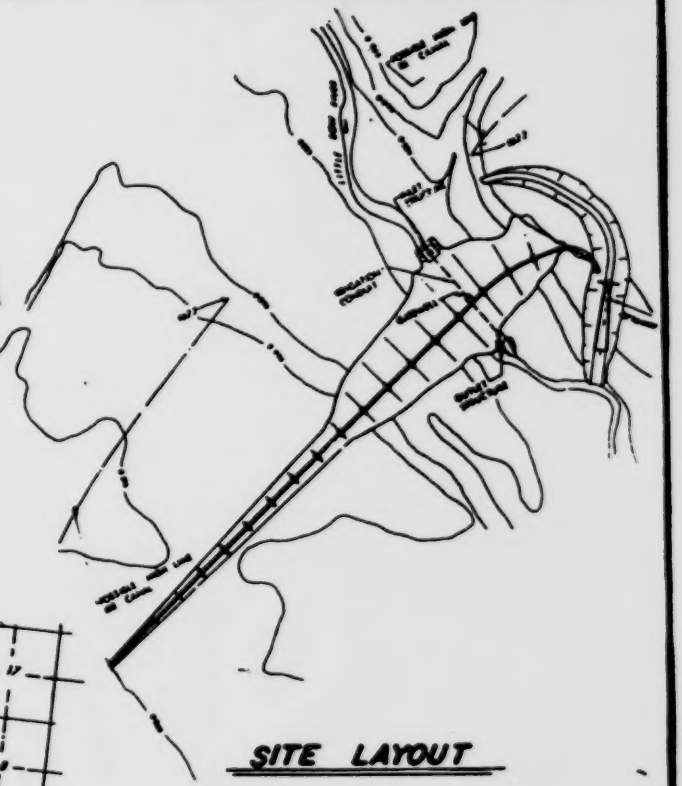


FLOODED AREA AND CAPACITY CURVES

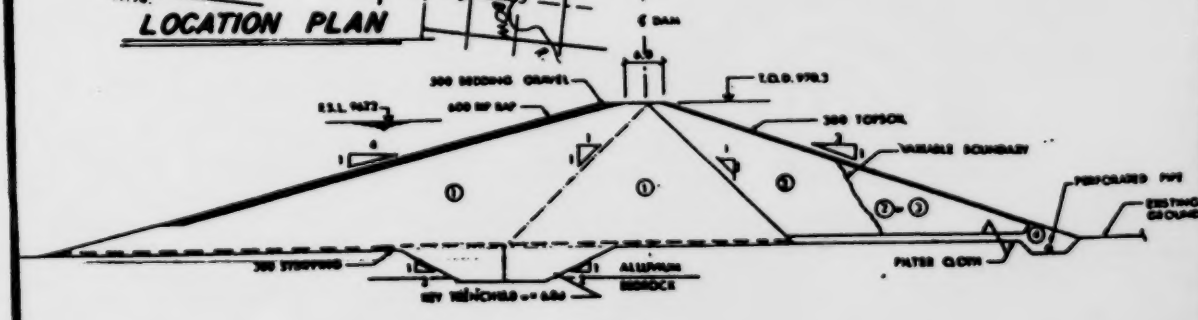


VALLEY CROSS SECTION AT DAMSITE

LOCATION PLAN



SITE LAYOUT



MAXIMUM EMBANKMENT SECTION

FIGURE. 5.

W-E-R ENGINEERING LTD.

LIST OF APPENDICES

A - WATER SUPPLY AND HYDROLOGY
(Nanuk Engineering and Development Ltd.)

B - GEOLOGICAL/GEOTECHNICAL EVALUATION
(Thurber Consultants Ltd.)

C - DETAILED SUMMARY OF DESIGN STUDIES AND COST ESTIMATES

D - RESERVOIR COSTS

LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
CONFLUENCE SITE (7B-1)
FEASIBILITY DESIGN REPORT
September, 1985

APPENDIX A

WATER SUPPLY AND HYDROLOGY STUDY

Box 817
Cochrane, Alberta T0L 0W0
Telephone (403) 932-3593

NANUK ENGINEERING & DEVELOPMENT LTD.

September 30, 1985

WER Engineering
Horizon 39 Building Box 2 Room 306
2735 39th Avenue NE
Calgary, Alberta
T1Y 4T8

Attention: Jack Wise

RE: LITTLE BOW RIVER BASIN
CONFLUENCE SITE 7B-1, FEASIBILITY STUDY

Dear Jack;

We are pleased to transmit our report on the hydrologic assessment of the confluence site 7B-1. The water supply provided by this reservoir and the flood flows affecting its design have both been addressed. The flood parameters should be regarded as preliminary only and must be substantially improved by additional hydrologic analysis prior to the preliminary design of a reservoir on the site.

PROFESSIONAL ENGINEER ALBERTA M. V. THOMPSON

Yours truly,

M. V. Thompson P.Eng.
Chief Engineer

MVT:lc
enclosure

PERMIT TO PRACTICE
NANUK ENGINEERING & DEVELOPMENT LTD.
Signature
Date Aug 30, 1985
PERMIT NUMBER: P 2906
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

1.0 WATER SUPPLY

The water supply available to the confluence site 7B-1 consists of natural flows in Mosquito Creek and the upper Little Bow River and diversion through Squaw Coulee and via the Little Bow Canal. The total basin yields provided by the three storage capacity alternatives - 30,000; 50,000 and 70,000 ac-ft. were calculated based on;

1. The water available for diversion from the Highwood River was calculated in accordance with the guideline permitting a total of 60% of the Highwood flow through both diversion canals with a minimum flow of 70 cfs downstream of the Little Bow Canal;

2. The physical capacity of the Squaw Coulee diversion was set at 60 cfs;

3. The capacity of the Little Bow diversion canal was varied from 100 to 500 cfs in 100 cfs increments;

4. The diversion canals were assumed to operate from April 15 to October 15 to provide water for irrigation and to the reservoir;

5. The municipal and riparian flow requirement of 15 cfs will be satisfied during the winter by diversion through the Little Bow Canal;

6. Natural flows were calculated on an annual basis and were assumed to enter the reservoir in April of each year;

7. The draft from the reservoir and Highwood diversions served a variable irrigation demand and a uniform municipal, instream use and residual flow requirement;

8. The seasonal irrigation demand was distributed in accordance with the calculated 1984 withdrawals as follows:

May	8%
June	30%
July	38%
August	19%
September	5%

9. Evaporative losses were calculated based on net evaporation at Calgary with a transfer factor of 1.0.

10. Water demands were distributed upstream and downstream of the reservoir site as follows except when upstream requirements exceeded 7,000 ac-ft. on either branch.

a. Mosquito Creek - 12%
b. Upper Little Bow - 20%
c. Lower Little Bow - 68%

11. Natural flows were calculated in the same manner as detailed in W.E.R. Engineering's report 'Little Bow River Basin, Storage Site Investigation'. The natural runoff in unit area terms are presented in Table I.

1.2 Methodology

The method of evaluation used to critical period 1916 to 1941 to calculate the water balance in the reservoir. The initial part of this period was used to check that the reservoir could be filled by the fall of 1928. The latter part represents the most critical drought within the period of record.

TABLE 1

NATURAL RUNOFF
(ac.ft./sq.mi./year)

YEAR	MOSQUITO CREEK	LITTLE BOW
1916	224	0
1917	263	0
1918	24	2
1919	8	12
1920	132	85
1921	14	15
1922	39	0
1923	144	2
1924	39	3
1925	51	12
1926	93	8
1927	225	31
1928	110	85
1929	114	8
1930	51	9
1931	0	12
1932	76	4
1933	55	13
1934	29	16
1935	21	9
1936	5	31
1937	7	7
1938	81	5
1939	25	12
1940	33	10
1941	16	12
MEAN	72	16

The diversions to the Little Bow Basin from the Highwood River were made in such a manner as to satisfy irrigation demand upstream of the confluence site. The firm supply available to both Mosquito Creek irrigators and those on the upper Little Bow is 7,000 ac-ft. on each reach. This firm supply is based on the distribution of seasonal irrigation demand stated previously and the acceptability of a 50% deficit in water supply in August (10% deficit in annual requirement) once in every 75 years.

The total basin yield provided by three storage levels at site 7B-1 was analyzed for canal capacities varying from 100 to 500 cfs in 100 cfs increments. The diversion guideline of 60% of the Highwood flow with a minimum residual flow in the Highwood of 70 cfs was used in the analysis.

1.3 Results

The results are presented in Figure A-1 in which variation in total basin yield with increasing capacity of the Little Bow Canal is graphed for three reservoir sizes. Observation from these results are;

1. At 30,000 ac-ft. storage capacity an increase in the diversion capacity in the Little Bow Canal above 300 cfs produce no increase in yield.

2. Increasing diversion canal capacity from 100 to 200 cfs increases basin yield by an average of 26% for all storage capacities.

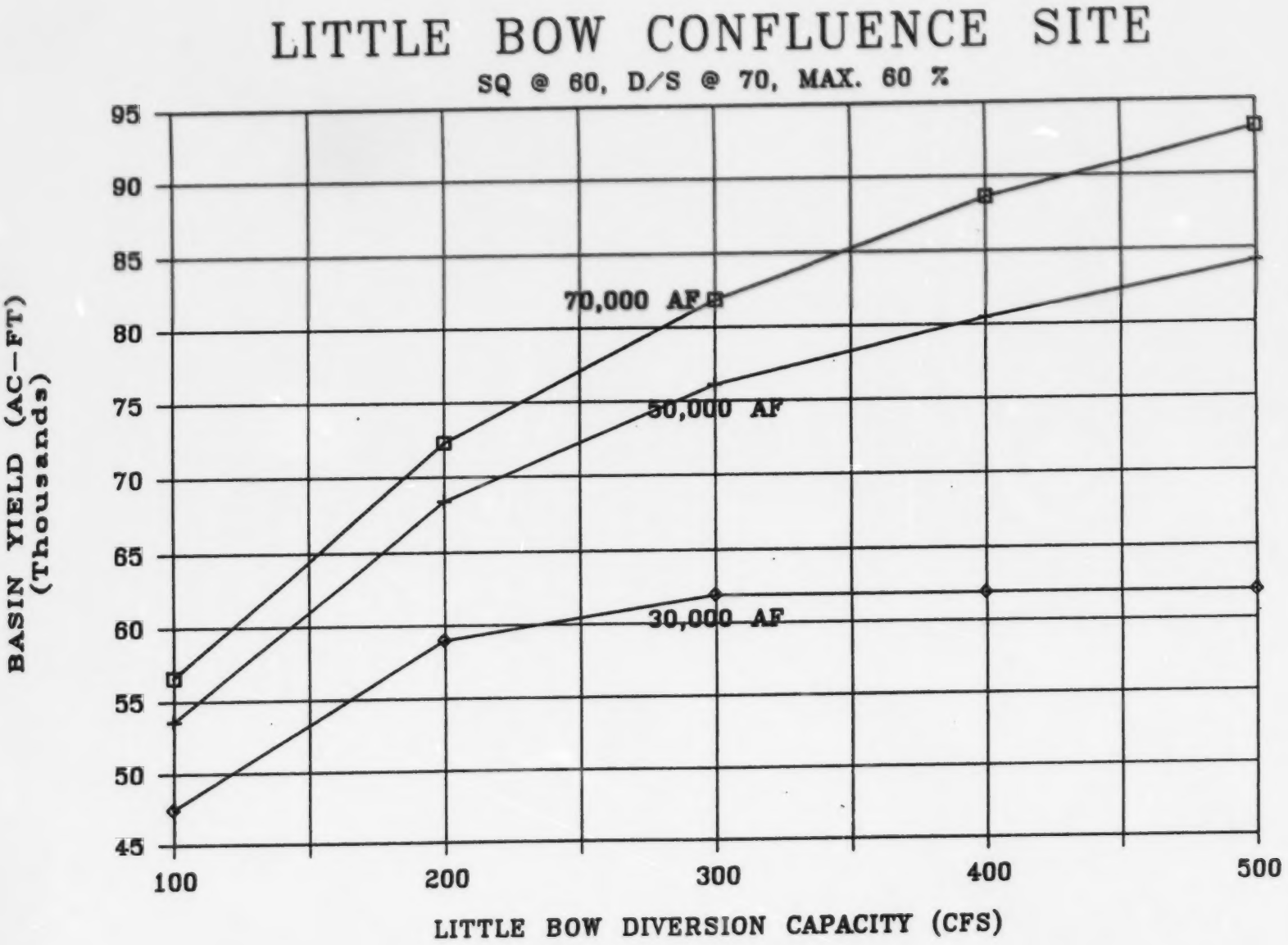

LITTLE BOW CONFLUENCE SITE
SQ @ 60, D/S @ 70, MAX. 60 %
BASIN YIELD (AC-FT)
(Thousands)
95
90
85
80
75
70
65
60
55
50
45
70,000 AF
50,000 AF
30,000 AF
100
200
300
400
500
LITTLE BOW DIVERSION CAPACITY (CFS)

FIGURE A.1

3. Increasing diversion canal capacity from 200 to 300 cfs increase basin yield by an average of 13% for the 50,000 and 70,000 ac-ft. storage sizes. The increased yield for the 30,000 ac-ft. reservoir is 5%.

4. The yield storage relationships at a diversion canal capacity of 300 cfs are;

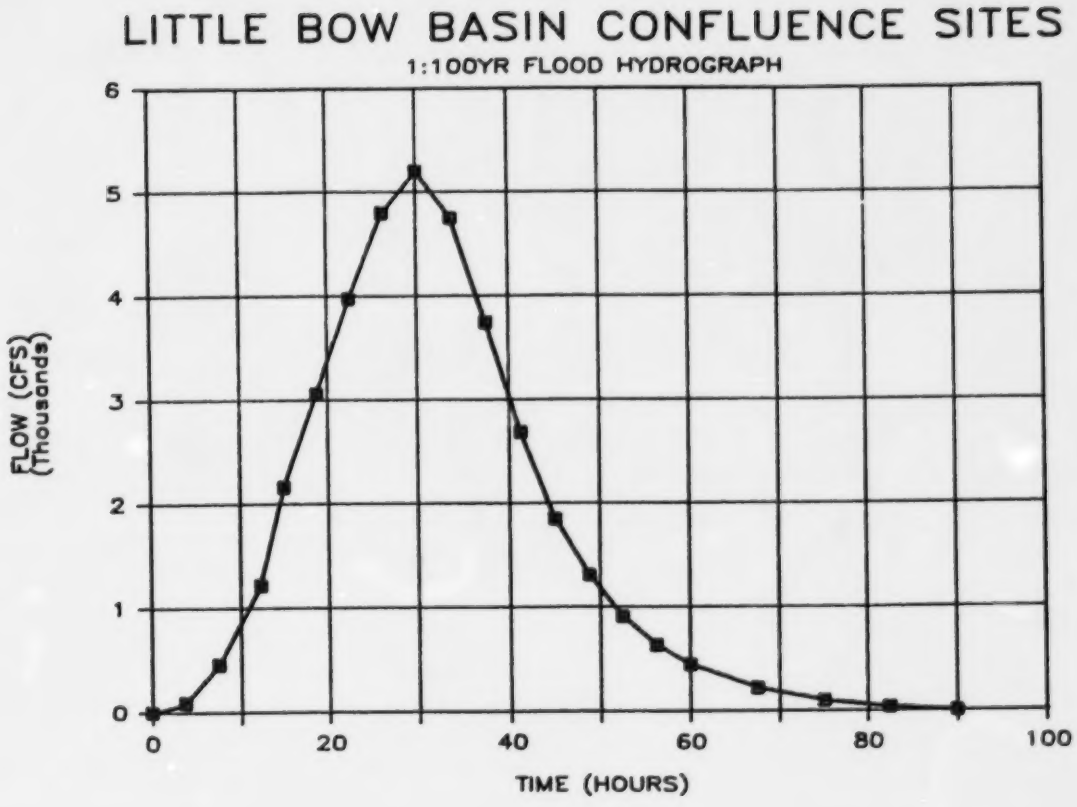
STORAGE ac-ft.	YIELD ac-ft.	Δ YIELD/ STORAGE	Δ YIELD/ Δ STORAGE
0	14,000*	-	-
30,000	62,000	1.6	1.6
50,000	76,000	1.24	0.7
70,000	82,000	0.97	0.30

* deficits in September in one year; 50% deficit in August, 1 year in 75

2.0 FLOODS

The probable maximum flood (PMF) was determined using the SNBB method for prairie streams. The 1 in 25 and 1 in 100 year floods and associated by hydrographs were based on frequency analysis of the recorded maximum mean daily flows of the Little Bow River at Carmangay. The peak instantaneous flows were calculated from the maximum mean daily flow using a historic ratio for recorded flood events.

The flood hydrographs representative of 0.5 of the PMF and the 1 in 100 year flood are presented in Figure A.2. The estimated values of the flood flows are listed below:


LITTLE BOW BASIN CONFLUENCE SITES
1:100YR FLOOD HYDROGRAPH
FLOW (CFS) (Thousands)
0
1
2
3
4
5
6
0
20
40
60
80
100
TIME (HOURS)

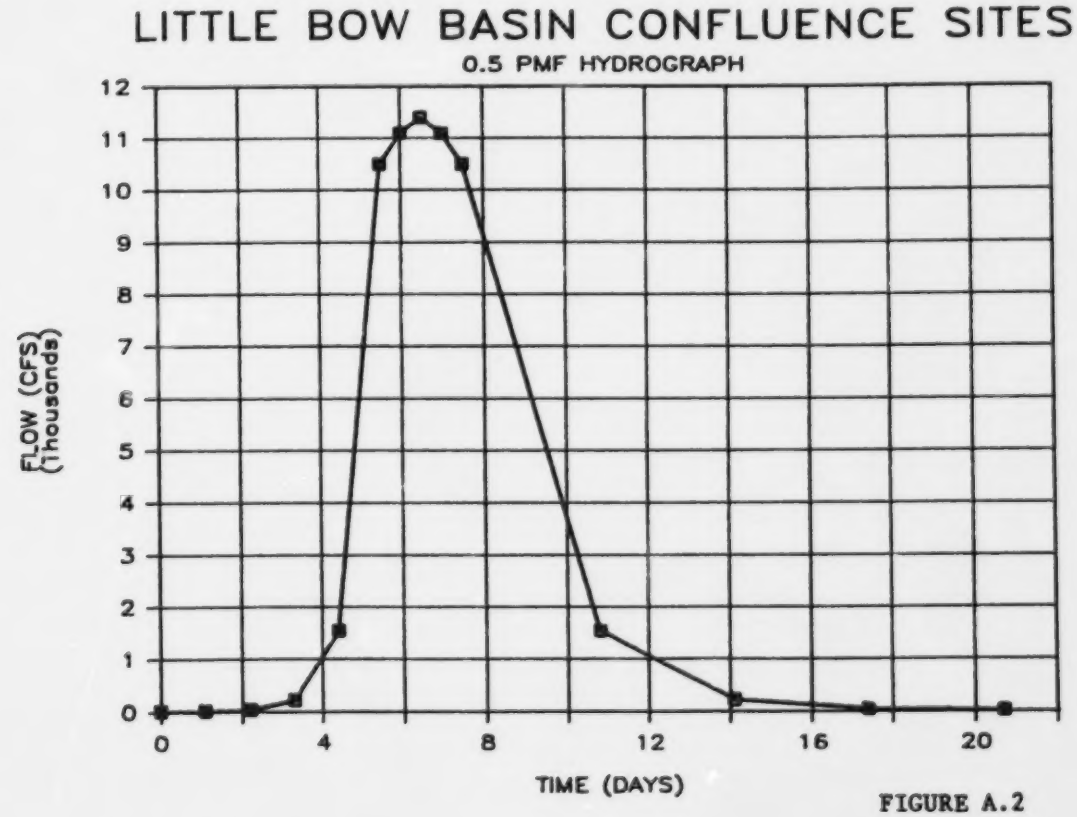

LITTLE BOW BASIN CONFLUENCE SITES
0.5 PMF HYDROGRAPH
FLOW (CFS) (Thousands)
0
1
2
3
4
5
6
7
8
9
10
11
12
0
4
8
12
16
20
TIME (DAYS)

FIGURE A.2

ESTIMATES OF FLOOD FLOWS

Drainage Area (MI2)	PMF Max. Mean Daily (cfs)	100 year Peak Instantaneous (cfs)	25 year (cfs)
455	20,000-25,000	5,200	3,000

The 1:25 and 1:100 year flows are considerably less than those previously estimated for Mosquito Creek, Site 4. The probable reasons for this difference are:

1. Mosquito Creek makes a major contribution to flood flows in the Little Bow below the confluence. Floods in Squaw Coulee, Nanton Creek and the upper Little Bow are relatively small. The combined flood flow, when routed, will produce a lesser peak than estimated for Mosquito Creek Site 4 due to attenuation, but the volume of the flood flow is increased.

2. Mosquito Creek flood flows are overestimated as a result of transposition from Willow Creek.

LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
CONFLUENCE SITE (78-1)
FEASIBILITY DESIGN REPORT
September, 1985

APPENDIX B

GEOLOGICAL/GEOTECHNICAL EVALUATION

LITTLE BOW RIVER BASIN STUDY
GEOTECHNICAL EVALUATION FOR
SITE 7B-1

Submitted

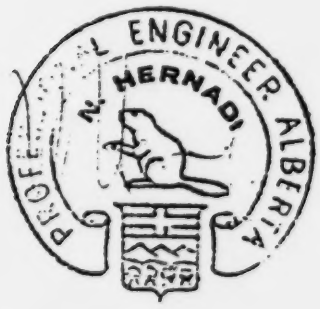
to

W-E-R ENGINEERING LTD.

Thurber Consultants Ltd.
Calgary, Alberta

PERMIT TO PRACTICE
THURBER CONSULTANTS LTD.
Signature
Date Sept 16, 1985
PERMIT NUMBER: P 355
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

L.B. Smith, P.Eng.
Review Engineer



September, 1985
File: 19-293-1

N. Hernadi
Project Engineer

SYNOPSIS

subject

This report presents a preliminary geotechnical evaluation of a damsite located on the Little Bow River about 20 km southeast of Parkland, Alberta. The site has been designated as Site 7B-1. Three different reservoir storage capacities are being considered at this site, and, therefore, 3 different embankment heights have been examined.

field work

A comprehensive drilling program was carried out by PFRA (1971) at the currently proposed damsite for the 86,300 dam^3 reservoir. For the 37,000 dam^3 and 61,700 dam^3 alternatives, the dam centreline has been relocated about 200 m upstream from the centreline assumed by PFRA. Subsurface information from the PFRA site has been extrapolated upstream for the preliminary geotechnical evaluation of the two smaller storages. If a decision is made to proceed with construction, a comprehensive drilling program should be carried out along the centreline finally selected.

site geology

The stratigraphy at the proposed damsite is relatively complex. At this location, the Little Bow River has cut down through about 20 metres of glacial clay till and about 10 metres into the underlying shale and sandstone bedrock. An alluvial terrace is present on the west (right) valley wall and the clay till in this area is overlain by 3 to 6 m of granular alluvium. Alluvial materials also overlie the clay till on the uplands to the east of the valley. This reach of the Little Bow River follows the preglacial Tee Pee Valley. As a result, at the proposed dam site, granular preglacial alluvium is present, sandwiched between the clay till and the bedrock surface.

reservoir shoreline stability

Three deep seated landslides have been noted along the valley walls upstream from the dam embankment. Detailed geotechnical investigations will be required to assess the effects of reservoir impoundment on the stability of these landslides. This work

should include an assessment of possible wave generation by landslide activity, and the risk of such a wave overtopping the dam embankment.

No significant concerns with respect to reservoir shoreline stability were identified in any other locations within the reservoir. Deep seated landslides are possible in some areas, however, the frequency of occurence of such landslides is expected to be very low. Shallow instability can be expected along the reservoir shorelines due to wave erosion. The beaches are expected to be self stabilizing over 20 to 30 years.

reservoir watertightness

Lateral seepage from the reservoir is not a major concern at this time. A detailed hydrogeological investigation will be required during detailed design in order to confirm this assessment.

embankment corss-section

A zoned earth fill embankment has been recommended for all 3 dam embankment heights. It is recommended that the downstream slopes are constructed at an average 3 horizontal to 1 vertical, while the upstream slopes would be constructed at an average 4 horizontal to 1 vertical. It is anticipated that the slopes can be bermed in order to reduce the quantity of fill required for the embankments. A decision in this regard will depend on the results of material balance studies, laboratory test results and stability analyses which should be undertaken during detailed design.

The failure mechanism of the three landslides located upstream from the proposed dam site should be established. The results of that work will have an impact on the soil parameters used for the design of the dam embankment. It is possible that the embankment sideslopes may have to be flattened significantly from those recommended here.

seepage below embankment

A conventional downstream drainage blanket and toe drain should be incorporated into the embankment design. Relief wells may be required at the downstream toe, however, more detailed subsurface information is required before a decision can be made in this regard. A compacted clay cut off must be used below the centreline of the embankment in order to prevent seepage through the alluvial sediments below the embankment. The cut off must be excavated through 7 to 10 metres of alluvium.

abutment seepage

Seepage around the abutments, through fractures and joints in the bedrock, is a concern at this site. Special prepartion of the abutments, as outlined in the report, will be required. It may be necessary to grout the rock at the abutments, however, further field investigations should be undertaken before a decision is made in this regard.

spillway

A single spillway, excavated around the east (left) dam abutment is being proposed at this site. The depth of excavation for the spillway will depend on the maximum reservoir operating level ultimately selected. The excavation will be carried out primarily through clay till and, in the case of the smallest reservoir, into the underlying bedrock. No significant construction difficulties with any of these excavations are anticipated. Blasting of cemented sandstone layers will be required if such material is encountered within the bedrock. Permanent slopes in clay till should be cut at 3 horizontal to 1 vertical. Permanent slopes in the bedrock should be cut at 2 horizontal to 1 vertical, primarily to minimize surface erosion of the shale. If steeper slopes are desired in the bedrock, surface treatment, such as shotcrete, should be considered.

low level conduits

The low level conduit should be founded either on bedrock, or on compacted silty clay fill which extends below the conduit

down to the bedrock. The conduit should therefore be located where the depth to bedrock is a minimum.

concrete

Sources for concrete aggregate have not been confirmed. It is anticipated that suitable aggregate (which will have to be crushed, screened and washed) can be located in the alluvial sediments upstream from the dam site. It is recommended that Type 50 sulphate resistant cement be used for all concrete in contact with soil at this site.

additional geotechnical investigations

Prior to final design, a comprehensive drilling program will be required in order to confirm subsurface conditions at the proposed site.

Other geotechnical studies, as detailed in the report, will also be required.

It is recommended that some phases of these geotechnical investigations be carried out at the pre-design stage.

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SECTION 1

INTRODUCTION

1.1 General

This report presents the results of a geotechnical evaluation of a dam/reservoir location designated as Site 7B-1, located in the vicinity of the Mosquito Creek-Little Bow River confluence in the Little Bow River Drainage Basin.

Authorization to proceed with the investigation was received from Mr. J.M. Wise, P.Eng., of W-E-R Engineering Ltd., and the scope of work for the evaluation based on discussions with Mr. Wise.

1.2 Project Background

Three potential damsites located in the Little Bow River Basin below the confluence of the Little Bow River and Mosquito Creek were evaluated in connection with a water storage investigation study undertaken by W-E-R Engineering Ltd. for Alberta Environment Planning Division. The results of this evaluation (including a preliminary geotechnical evaluation) are presented in a conceptual design report which was submitted to Alberta Environment (W-E-R Engineering Ltd., 1985). The report concluded that of the three sites studied, Site 7B-1 had the best overall potential for water storage. The report recommended that more detailed engineering studies be undertaken for Site 7B-1.

SECTION 2

METHOD OF INVESTIGATION

2.1 Review Background Information

All available published and unpublished information concerning the geology and hydrogeology for Site 7B-1 was reviewed. Specifically, reports by PFRA, Alberta Environment and W-E-R Engineering Ltd. were the major sources of data. A list of the sources of information referred to is presented at the end of the text of this report.

A geotechnical investigation was undertaken by PFRA at Site 7B-1 in 1971, and this report provided valuable information concerning the subsurface conditions at this site, since test holes were not drilled in conjunction with the current investigation.

2.2 Engineering Terrain Analysis

An engineering terrain analysis of the site was carried out using high and low level airphoto coverage, to assess the geological setting and its implications on construction of the proposed dam.

The analysis include the mapping of surficial soils at the damsite and within the reservoir, identification of slope instability, and the delineation of surface drainage patterns and other pertinent features.

SECTION 3

REGIONAL GEOLOGY

3.1 Physiography, Relief and Topography

The study area is situated, within the Alberta Plain physiographic division of the Interior Plains, some 20 km southeast of Parkland, Alberta, as shown on Drawing 1, Appendix A.

The terrain within the area generally slopes gently from west to east. Elevations average about 950 m above sea level (asl). Superimposed onto the gently sloping plain is a series of ice-marginal melt-water channels. These generally trend northwest-southeast and are incised to depths of 15 m to 30 m below the general prairie level within the study area.

In the vicinity of the Little Bow River valley, relief is generally gently undulating to rolling, with local areas of subdued hummocky terrain. While slopes of less than 15 degrees are characteristic, slopes in excess of 50 degrees exist along the walls of the various meltwater channels. A short distance to the west, around Clear Lake, areas of ridged to hummocky terrain are present, developed on glaciofluvial esker and kame deposits.

3.2 Bedrock Geology

Although Green (1982) indicates that Upper Cretaceous Horseshoe Canyon Formation bedrock underlies the confluence area, previous drilling results at this site suggest that the overlying Willow Creek Formation, of Upper Cretaceous to Tertiary age, is the uppermost bedrock unit within the study area. This formation, a lateral stratigraphic equivalent of the Paskapoo Formation, comprises an interbedded sequence of non-marine shales and siltstones, with some medium to coarse-grained sandstones. Minor constituents are bentonitic shale and bentonite.

The bedrock is usually masked by younger Pleistocene and Holocene sediments. Locally, it is exposed, or present beneath a shallow colluvial veneer, along the walls of the various meltwater channels.

3.3 Buried Valleys

In pre-glacial times, an extensive network of river and stream valleys was developed in this part of Alberta.

These were cut down into bedrock and have subsequently been infilled by a thick overburden sequence that may include a thick basal sand and gravel unit. Within the area of interest, the Tee Pee Valley, the thalweg for which is indicated on Drawing 1, Appendix A, is most important. It is inferred to be followed by the Little Bow River valley throughout the reach of interest, with a tributary joining it from the west a short distance downstream from the proposed dam site.

3.4 Surficial Geology

Surficial deposits, of both glacial and post-glacial origin are present in the vicinity of the confluence of the Little Bow River and Mosquito Creek.

Glacial till, forming ground moraine, is the dominant surfical material. It comprises a heterogeneous mixture of clay to gravel size material, deposited by Laurentide ice as it traversed the area (ground moraine) or from ice that decayed in place (hummocky moraine). Glaciofluvial deposits, sands and gravels predominantly, deposited by meltwater, are also locally abundant, notably in the Clear Lake area to the west of the Little Bow River valley, where they form esker, delta and kame deposits. On the uplands adjacent to the river valley, sands and silts of glaciolacustrine (glacial lake) origin frequently form a veneer over the morainal deposits.

Since deglaciation, post-glacial erosional and depositional processes have led to the development of a variety of more recent deposits. Alluvial and colluvial materials are most important, respectively forming floodplain and terrace, and slopewash deposits along the floors and walls of the various meltwater channels. Recent lake deposits and organics have accumulated locally in terrain depressions.

The thickness of the surficial deposits overlying the bedrock averages in the order of 10 to 20 m over much of the study area. Within the buried pre-glacial Tee Pee Valley, however, thicker surficial deposits may exist.

3.5 Surface Drainage and Groundwater

The area of interest is drained by a series of generally northwest to southeast trending streams, of which the Little Bow River and Mosquito Creek are amongst the most important. Surface drainage and run-off is generally into these water courses, but also locally into closed depressions whence it either evaporates or seeps into the ground.

Three groundwater flow regimes, one superimposed on the other, are identified. On a "regional" scale, groundwater flows at depth, from recharge areas in the Foothills and Rockies to discharge further east in the Plains. On a smaller scale, "local" groundwater flows are recharged in the higher sections of the area and discharge into the river and creek valleys. Finally, "perched" flows exist within granular, and more permeable, overburden zones and at the overburden-bedrock contact. These latter discharge into terrain depressions, as well as the river and creek valleys.

SECTION 4

GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

4.1 General

Little Bow Site 7B-1 is located approximately 20 km southeast of Parkland, Alberta, as shown on Drawing 1, Appendix A. The area of the proposed dam site is located in Section 5, Township 15, Range 27, West of the 4th Meridian.

It is understood that three possible storage capacities are being considered, with the following parameters:

Dam Capacity (dam^3)	Full Supply Level (m)	Freeboard (m)	Top of Dam Elevation (m)	Flooded Area (ha)	Normal Drawdown Range (m)
37,000	961.2	2.9	964.1	656	5
61,700	964.7	3.0	967.7	862	7
86,300	967.2	3.1	970.3	1075	9

The maximum embankment height will range between about 18 and 24 m, and the crest length between 465 and 1170 m. The difference in lengths consist mostly of a relatively low dyke required on the west side of the dam on a high level terrace to provide the required full service level for the larger storage capacities.

As indicated above, the normal operating drawdown ranges from 5 to 9 metres depending on the reservoir storage capacity being considered. Maximum live storage drawdown which may occur once per 25 years, would be to the top of the low level inlet for each of the three cases.

A concrete chute spillway would be constructed around the east abutment of the embankment, and a 1.5 m diameter precast pipe low level conduit with invert elevation of 945 m would be located below the east half of the embankment.

4.2 Reservoir Geology

The surficial geology of the reservoir is shown in Appendix A, Drawing 2. The reservoir outline for the largest proposed storage capacity (86,300 dam^3) is shown on this drawing. In each instance, ground moraine, consisting of silty clay and generally veneered by glaciolacustrine silt

and sand, is the near surface soil on the uplands adjacent to most of this reach of the Little Bow River. Locally, high level alluvial terraces are present also, consisting of sand, silt and minor gravel, underlain by clay till. As indicated on Drawing 2, the valley bottom is infilled with recent alluvial deposits, consisting of silts, sands and gravel.

The Little Bow River valley has been cut down through glacial till into the underlying bedrock. Bedrock is exposed in some locations along the valley walls. In most locations however, it is covered by a thin veneer of colluvium (slopewash).

The valley walls throughout the area of interest are relatively steep, ranging up to 45 degrees, and generally stable. Evidence of deep seated slope instability is apparent, however, on the walls of the Mosquito Creek valley and along the Little Bow River approximately 9 km northwest of the dam site (Drawing 2, Appendix A). Shallow, localized instability of the colluvial materials also occurs in a few locations, particularly where the Little Bow River and Mosquito Creek are eroding directly at the toe of the valley walls.

4.3 Reservoir Shoreline Stability

Impoundment of the reservoir will cause a decrease in the overall stability of the valley walls mainly due to rapid drawdown and wave erosion. In addition, increased irrigation on the uplands adjacent to the reservoir will also tend to decrease the stability of the slopes. These effects may be significant, depending on local topography, however, the valley slopes should remain generally stable.

The stability of the deep seated large scale landslides, identified within the reservoir is of concern. A detailed evaluation of the risk and consequences of instability and these locations should be undertaken during detailed design. This work should include an evaluation of possible wave generation by landslide activity and the risk of such a wave overtopping the dam embankment.

The limits of any new deep seated slope instabilities which occur following reservoir development are expected to be confined within a zone described by a line drawn parallel to the valley at a distance of about 30 m back from the top of the valley walls. This setback should be reviewed for site specific cases.

The reservoir full supply level is located below the elevation of the bedrock surface for the 37,000 dam^3 reservoir, but generally above it for the other two alternatives. For the smallest reservoir, the shoreline will erode into thin colluvium which is underlain by shale and sandstone bedrock. It is expected that small scale, localized instability of the slopes will occur due to wave erosion along the shoreline, particularly in locations where uncemented sandstone is present at the shoreline. This instability will be confined to the range of drawdown in the reservoir. The instability is expected to be self stabilizing as beaches develop over a period of 20 to 30 years.

Where the full service level is above the overburden-bedrock contact, as is the case for the medium and largest reservoir, the shoreline will erode into thin colluvium which is generally underlain by clay till. It is expected that small scale localized instability will occur along the shoreline, particularly in locations where sand is present. The instability will be confined to the range of drawdown in the reservoir, and are expected to be self stabilizing as beaches develop over a period of 20 to 30 years.

4.4 Reservoir Watertightness

Detailed information concerning the present ground water elevation adjacent to the proposed reservoir is not available. Information from small scale hydrogeological maps for this area indicate, however, that the groundwater table elevation is about 975 m. Assuming reservoir elevations of 961.2 to 967.2 m for the three proposed capacities, it can be expected that the reservoir will not have a significant effect on the groundwater table in the area, except in a peripheral zone.

Some lateral seepage into or out of the valley walls will undoubtedly occur, primarily through joints and fractures in the bedrock, as well as through the more pervious clean sandstone beds. However, in view of the anticipated existing high regional groundwater table, water leakage from the reservoir into the valley walls is not expected to be significant. More detailed information concerning existing groundwater levels in the vicinity of the reservoir should be obtained in order to confirm this assessment.

A buried valley (Tee Pee Valley) has been mapped upstream of the proposed damsite (Geiger, 1968). The approximate location of this valley is shown on Drawing 1,

Appendix A. No information concerning the nature of the valley infill at its intersection with the proposed reservoir is available. Assuming the groundwater table within the preglacial valley is consistent with data from the small scale hydrogeological maps for this area, it is speculated that no significant seepage losses into the valley will occur. It is recommended, however, that a detailed subsurface investigation be undertaken in order to establish the stratigraphy within the buried valley, before a final decision is made to construct a dam at this site.

4.5 Damsite Geology

A preliminary geotechnical investigation of Site 7B-1 was undertaken by PRFA (1971). The drilling was carried out at the currently proposed damsite for the 86,300 dam^3 reservoir. The dam centreline for the 37,000 dam^3 and 61,700 dam^3 alternatives is located about 200 m upstream of the site previously drilled. The subsurface information from the PFRA investigation is used for both sites for purposes of this preliminary study, however detailed drilling will be required to confirm subsurface conditions at the site finally selected. For convenience, a brief summary of the results of the PFRA investigation is presented here. More detailed information can be obtained from the report prepared by PFRA.

4.5.1 Surface Conditions

The distribution of surficial deposits and other pertinent features at Site 7B-1 is presented on Drawing 3 in Appendix A.

At this site, the Little Bow River flows in a southerly direction. The river is located in the centre of a U-shaped valley which has a base width of about 300 m. Abutment side slopes at the centreline are between 3 horizontal to 1 vertical and 4 horizontal to 1 vertical. The main valley is located about 27 m below the level of the east upland. A flat lying terrace, roughly 600 m wide, is located between the 21 m high west abutment of the main valley and the west upland. Natural valley slopes in the vicinity of the site vary from 1.5 horizontal to 1 vertical, to 4 horizontal to 1 vertical or flatter. All slopes over 15 m in height average 2 horizontal to 1 vertical or flatter, whereas some slopes under 15 m in height are as steep as 1.5 horizontal to 1 vertical. Bedrock is exposed in a 3 to 7 m near-vertical face at the toe of the east abutment about 250 m downstream of the proposed dam centreline for the 86,300 dam^3 reservoir.

The valley slopes near the dam site are mantled by slope wash or slump debris generally masking the underlying soils. The valley bottom contains recent alluvial deposits of varying texture.

Surface deposits on the high level terrace on the west side and the east uplands are comprised predominantly of alluvium consisting of a wide range in texture.

4.5.2 Subsurface Conditions

Subsurface conditions at the damsite are presented in Drawing 4, Appendix A. This drawing is a simplified version of a detailed subsurface cross-section at the damsite presented in the PFRA report. The following major soil units were encountered at the site:

- Alluvium (east upland, west high level terrace, valley bottom)
- clay till
- interglacial deposits
- bedrock

East Upland Alluvium

A variable thickness of alluvium consisting of pervious gravelly sands to impermeable silts and clays mantles the east upland. These deposits may represent a seepage hazard due to their permeable nature adjacent to the east abutment. Average properties for this material are summarized in Table 4.1

West High Level Terrace Alluvium

Between the west abutment and upland is a relatively flat, 600 m wide, high level terrace containing 3 to 6 m of silty sands and gravels. The alluvium varies from pervious to impervious with most materials being moderately impermeable. Average properties for this material are summarized in Table 4.1.

Valley Bottom Alluvium

As indicated on Drawing 4, the valley bottom has been infilled with 4 to 8 m of extremely variable alluvial materials. The eastern portion of the flood plain is about 4 to 5 m thick, whereas west of the river it is 6 to 8 m thick. The alluvium is basically of two types

- impervious silts and clays, and granular materials covering the entire range of permeability from pervious to impervious.

One unconfined compressive strengh test on a sample of alluvial clay was reported at only 90 kPa. Reported pocket penetrometer values indicate unconfined compressive strengths for this material ranging between 70 kPa to 400 kPa, with an average of 220 kPa for 5 determinations.

Average properties for the valley bottom alluvium are summarized in Table 4.1

Clay Till

Clay till overlies bedrock on the east upland and an interglacial deposit on the west upland. The till deposits are predominately low to medium plastic clay with some sand pockets, lenses and layers. On the east upland the till could exceed 20 m in thickness, and underlies a zero to 3 m thick mantle of surface alluvium. On the west upland, the clay till is 8 to 10 m thick and is overlain by 3 to 7 m of high level terrace granular materials.

A summary of properties for the clay till is given in Table 4.1

Interglacial Deposits

As indicated on Drawing 4, a 7 m thick stratum of interbedded and laminated alluvial clays to sands is sandwiched between the overlying glacial clay till and the bedrock, in the area below the west terrace. For purposes of this report these alluvial materials are designated as interglacial deposits, but could be preglacial. These materials are believed to be associated with the pre-glacial Tee Pee Valley. The deposit is characterized by a great number of slicked surfaces, which could be old failure surfaces.

Five consolidated undrained triaxial strength tests at various confining pressures were conducted by PFRA on samples with similar weak planes. These limited test results indicate that the preglacial clays have the following strength parameters:

	Cohesion Intercept	Effective Angle of Friction
Peak Strength	20 kPa	28°
Residual Strength	0 kPa	19°

It should be noted that the residual angle of friction for the clay may be much lower than 19o, if the tests were carried out to larger strains than is possible in the triaxial test.

A summary of other pertinent properties of this material is given in Table 4.1.

Bedrock

The bedrock at the site is the Willow Creek Formation of Upper Cretaceous Age consisting perdominately of interbedded silty shale, which contains some organic and bentonitic facies, minor silty to calcareous sandstone layers and numerous limestone nodules and concretions. Bedding is essentially horizontal and the different bedrock strata grade almost imperceptively from one rock type to another. Generally the bedrock is hard and dense, however, there are noticeably softer and harder sections, dependent on composition, degree or amount of cementation, and weathering. In general, the calcareous sandstones and shales are appreciably harder than the other bedrock. The cementation of the bedrock varies from poorly to well cemented, with the sandstones generally being better cemented than the shales. In slaking tests conducted by PFRA most of the shales at the site broke down into gravel or colloidal sizes when immersed in water for 20 to 30 minutes. The bedrock is light grey to brown depending to some extent on the degree of weathering.

In addition to differences in composition of the bedrock, there is a considerable variation in water contents, densities, Atterberg Limits, sand and clay content, hardness and cementation even within similar bedrocks. As a result, materials with considerably different stress-strain and strength properties are often adjacent to one another. This difference is greatest between the harder, well cemented, calcareous sandstones and shales and the adjacent softer shales.

The index tests indicate that the bedrock of the Willow Creek Formation is relatively competent. An exception is the few bentonitic facies within the formation which are less competent and in which high pore pressures, slope instability, swelling and rebound problems may occur. These bentonitic materials are believed to be of only limited extent and no evidence was obtained that continuous layers occur.

A summary of bedrock properties is given in Table 4.2.

TABLE 4.1

SITE 7B-1, SUMMARY OF SOIL PROPERTIES

Property	Unit	East Upland Alluvium		West Terrace Alluvium		Valley Bottom		Clay Till	Interglacial Alluvium
		Granular	Fine	Granular	Fine	Granular	Fine		
Wet Density	kg/m^3	-	1955	-	1603	1939	1859	2179	2099
Water Content	%	-	15	4	11	14	19	15	19
Liquid Limit	%	-	24	-	-	-	35	29	45
Plastic Limit	%	-	15	-	-	-	20	14	18
Plasticity Index	%	-	9	-	-	-	15	17	27
Fines < 0.074 mm	%	8	21	10	38	21	-	-	81
Activity	-	-	-	-	-	-	-	-	0.66
Pocket Penetrometer*	kPa	-	234	-	579	-	-	517	407
Water Content at Standard Proctor Maximum Density	%	-	-	-	-	-	-	13.2	-

* unconfined compressive strength

NOTE: Values for soil properties given above are average values as reported by PFRA (1971), converted to metric units where applicable.

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TABLE 4.2

SITE 7B-1, SUMMARY OF ROCK PROPERTIES

Property	Unit	Shales	Sandstones
Wet Density	kg/m3	2131	2112
Water Content	%	14	9
Liquid Limit	%	45	-
Plastic Limit	%	23	-
Plasticity Index	%	22	-
Fines 0.074 mm	%	96	48
Activity	-	0.75	-
Pocket Penetrometer*	kPa	675	1033
Lab Qu *	kPa	186	-
Secant Modulus	MPa	10.3	-

* unconfined compressive strength

NOTE: Values for rock properties given are average values as reported by PFRA (1971), converted to metric units where applicable.

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Groundwater

The PFRA data indicates that the groundwater table below the east upland is within 1.5 to 3 m of ground surface. Below the high level terrace on the west side the groundwater table is at, or near ground surface. The presence of some springs in the area was also reported by PFRA.

The water table in the valley bottom is at about river level.

No artesian water pressures were reported in any of the test holes drilled by PFRA into bedrock at this site. Data on piezomatic elevations within the impervious bedrock foundations at this site are not available. A detailed hydrogeological assessment of the site has not been undertaken.

4.6 Dam Embankment

4.6.1 Siting

There are no significant geotechnical factors affecting the positioning of the embankment centreline at this site. It should be recognized however, that with the embankment for the 37,000 dam^3 and 61,700 dam^3 reservoirs moved approximately 200 m upstream from the location proposed by PFRA, it will be necessary to drill additional test holes on the new embankment centreline in order to confirm the stratigraphy at that location. It is possible, but not likely, that subsurface conditions at any new location may be significantly different from those encountered by PFRA.

In view of the preliminary nature of the PFRA investigation, additional drilling and test pitting will also be required at the currently proposed damsite for 86,300 dam^3 to further delineate subsoil conditions and to identify potential difficulties in making deep excavations.

A small ravine is located on the east side of the valley a short distance upstream from the proposed dam centreline of the 37,000 dam^3 and 61,700 dam^3 reservoirs, as shown on Drawing 3. If the embankment centreline is moved further upstream, measures may be required to prevent surface water from this ravine from eroding the downstream face of the embankment.

4.6.2 Embankment Configuration

Tentative configurations for the proposed dam embankment are presented in Appendix A, Drawing 5, 6 and 7 for the 3 proposed storage capacities. It is expected that the configurations presented on these drawings would be refined and optimized during detailed design, as a result of material balance studies and detailed slope stability analyses.

An average upstream slope of 4 horizontal to 1 vertical is recommended, with an average downstream slope of 3 horizontal to 1 vertical. Both the upstream and downstream slopes may be bermed in order to reduce the volume of material required. A decision in this regard would be made during the detailed design phase.

As mentioned earlier, a number of deep seated landslides exist on Mosquito Creek upstream from the proposed dam site. These landslides are believed to be seated in clay till, however, this has not been confirmed. If the landslides prove to be seated in bedrock, this would indicate the bedrock may contain pre-existing failure surfaces. If this is the case, then bedrock core would have to be obtained at the dam site. If numerous failure surfaces are found in the core, then the embankment slopes may have to be flattened significantly from those recommended here.

The upstream portion of the dam (Zone 1) would be constructed of low to medium plastic clay till. This material would be obtained from the upper portions of the spillway excavation and from the uplands. The clay till is at or slightly above optimum water content and it may not be necessary to add water during compaction.

The downstream portion of the dam (Zones 2 and 3) would be constructed of random semi-pervious to pervious sands and gravels taken initially from the cutoff excavation and possibly the spillway excavation. Additional material for this portion of the dam would have to come from borrow sources located on the high level terrace and the east uplands. The coarser and more pervious materials should be placed towards the downstream face of the dam.

Compacted shale and sandstone bedrock, if encountered in the spillway excavations, could also be used in the downstream portion of the dam as indicated on the drawing. As noted, the exact boundaries of the

various zones can be modified to make optimum use of the available materials.

Clean sand and gravel would be placed to form the toe drain, as shown on the drawings. This material may be available within the alluvium, however it may be necessary to screen and wash the pit run gravels in order to obtain material meeting the required specifications. In addition, a number of potential commercial sources of granular materials were identified by PFRA for this material. Filter cloth should be used, as indicated on the drawings, to prevent migration of fines into the toe drain.

Preliminary calculations indicate that the dam foundation can be expected to settle in the order of 0.3 m or so during construction. The dam crest would settle an additional 0.15 m or so when the reservoir is filled. It may be desirable to raise the finished dam crest above the design elevation in order to accommodate movements of this order. If required, a detailed settlement analysis of the embankment can be undertaken during detailed design.

4.6.3 Seepage Control

All organic material and all soft or loose inorganic materials should be stripped from below the area of the embankments. The entire area should be scarified and recompacted to a depth of at least 0.3 m. Proof rolling is recommended prior to placing fill in order to identify any soft zones. Waste soil should be placed upstream of the dam preferrably in the existing river channel.

Pervious layers within the alluvium represent the most serious source of seepage beneath the embankment. A positive cutoff, extending into the shale bedrock below the embankment is recommended. The width of the base of the cutoff could vary from a minimum 9 m to 15 m (or greater). The width of the cutoff could be increased, as desired, in order to obtain the required quantities of gravel for use in the downstream toe drain. Seepage into the cutoff excavation is a concern. It is recommended that a number of test pits be excavated along the centreline of the cutoff in order to provide a better assessment of the behaviour of excavation sideslopes.

Seepage through fractures and joints in the bedrock below the dam is a concern. It may be necessary

to extend the toe drain down into the bedrock in order to relieve potential uplift pressures at that point. Alternatively, pressure relief wells should also be considered. The requirement in this regard will depend on the bedrock stratigraphy below the downstream toe.

Abutment seepage is a concern. The PFRA data indicate a soil stratigraphy comprised of slope wash overlying alluvial material and clay till, with the possible presence of interglacial alluvial deposits in the west abutment area. The east abutment is comprised of slope wash over alluvial and glacial till deposits overlying bedrock. The precise requirements to control this source of seepage cannot be made until data from additional field permeability tests in the bedrock and overburden soils is obtained.

Within the overburden soils in the abutment areas it will be necessary to extend the positive cutoff into the clay till and interglacial alluvium.

As previously mentioned, the presence of the buried Tee Pee Valley in this area, which may be located under the west abutment area, is a concern with respect to seepage, and a detailed subsurface exploration would be required to establish the stratigraphy of the buried valley in this area.

The bedrock in the lower portion of the east abutment areas of the dam would likely be stepped, in order to facilitate the placement and compaction of fill against the abutments. Seepage along the interface between the compacted clay fill and the bedrock may require shotcreting of the bedrock surface in that area where the impervious clay core is placed against the abutment. Clay till compacted against the abutments should be compacted at 2 or 3% wet of optimum. Further, a clean gravel layer should be placed against the bedrock surface immediately downstream from the impervious zone. This gravel layer should be connected to a pipe and, if practical, discharged to a location where it can be monitored. The purpose of the gravel layer is to safely drain any seepage water from the bedrock and the bedrock contact.

4.6.4 Upstream Slope Protection

The upstream face of the dam must be armoured within the normal operating reservoir levels in order to reduce possible wave erosion. As indicated on

Drawings 5, 6 and 7, the slope protection should consist of 0.5 m of well graded sand and gravel covered by 0.6 m of rock armour. The class of rock armour required will be established during detailed design.

A suitable source of rip rap slope protection has not been established.

An often acceptable alternative to expensive rip rap is to use coarse pit run gravel and accept the need for occasional regrading of the slopes. The use of soil cement could also be considered.

4.6.5 Downstream Slope Protection

Measures will be required to prevent erosion of the downstream face of the embankment due to surface runoff. The crest of the dam should be surfaced with gravel (for trafficability) and sloped towards the reservoir. The downstream face of the embankment should be covered with 0.3 m of topsoil and hydro-seeded.

4.7 Spillway

Construction of the spillway approach channel around the east abutment will require an excavation mostly through clay till. Some excavation into the bedrock may be required for the lowest of the 3 embankment configurations being considered. Further investigation will be required to identify potential water bearing layers and details of bedrock stratigraphy along the proposed spillway alignment.

No major difficulties with the spillway excavation are anticipated. It is expected that the shale and the weaker sandstones which may be encountered in the lower portion of the spillway excavation can be removed by ripping. Some blasting of cemented sandstone layers may be required. Occasional seepage zones may be encountered, however water quantities are expected to be low and water can be collected into drainage ditches and sumps using conventional construction practice.

Permanent cutslopes in the clay till should be no steeper than 3 horizontal to 1 vertical in order to ensure long term stability. Flatter sideslopes within the clay till may be desirable in order to liberate more material for use in the construction of the embankment.

Permanent cutslopes within the shale or sandstone bedrock could be trimmed at 1 horizontal to 1 vertical or steeper and they would remain stable. However, the exposed

shale is susceptible to weathering and erosion which could become a major maintenance problem. For this reason, it is recommended that permanent slopes in the bedrock be cut no steeper than 2 horizontal to 1 vertical in order to minimize surface erosion. If steeper slopes are desired, surface treatment, such as shotcreting, should be considered.

It is understood that the spillway chute will be lined with concrete in order to prevent erosion. It is understood that the gradient for the spillway chute cannot be steeper than 3 horizontal to 1 vertical for hydraulic reasons. This slope will be satisfactory in the shale bedrock from a slope stability point of view. If necessary, steeper longitudinal slopes could be considered. In any event, it will be necessary to anchor the spillway chute to the slope, and install suitable drainage below the chute to prevent uplift due to water pressure.

It is understood that the concrete weir and gate structure at the top of the spillway chute will be subjected to horizontal loads, which must be transferred to the underlying soils. For preliminary design of the weir, a nominal angle of friction of 20 degrees may be used for the clay till subsoil. This value should be confirmed during detailed design.

The shale bedrock can be expected to rebound due to the weight of material removed from the excavation. Rebound will be primarily elastic and will therefore take place as the excavation proceeds. Past experience with deep excavations in materials similar to those encountered here, indicates that all rebound (including time dependent rebound) will be essentially complete 2 to 3 months after the excavation is completed. The total rebound will range from 30 to 80 mm. A more precise estimate of the rebound can be made once the configuration and depth of the excavation has been confirmed.

4.8 Low Level Conduit

In view of the highly variable compressibility of the alluvial sediments at this site, it is recommended that the low level conduit be founded in bedrock. If this is not possible due to elevation restrictions, it is recommended that all alluvial material below the low level conduit through Zone 1 be subexcavated down to bedrock, and replaced with well compacted clay till. In either case it is desirable to place the conduit in a location where the depth of bedrock is at a minimum. The subsurface information reported by PFRA indicates that the most suitable location is on the east side of the river channel.

It is recommended that cutoff collars be cast around the upstream portion of the conduit in Zone 1 in order to reduce the possibility of seepage. The downstream portion of the conduit through Zones 2 and 3 should be surrounded by a minimum thickness of 1 m of pervious filter in order to prevent possible erosion along the conduit.

Settlement and horizontal elongation of the conduit can be expected as the embankment fill is constructed. The conduit must be designed to accommodate these movements. The deformation could be minimized if the conduit were placed directly on bedrock.

4.9 Concrete

A suitable source of concrete aggregate has not been identified. The alluvial deposits in the valley would have to be crushed, screened and washed in order to obtain suitable aggregate for concrete. The durability and soundness of these aggregates is not known.

Unless water soluble sulphate tests are undertaken on soil samples during final design it is recommended that Type 50 sulphate resistant cement be used for all concrete in contact with soil at this site. This is particularly important for the low level conduit, which is subject to an unfavourable hydraulic environment from the point of view of potential attack by sulphates.

4.10 Additional Geotechnical Investigations

If a decision is made to proceed with construction of a dam at Site 7B-1, the following geotechnical work is recommended:

1) Test pits and test holes should be drilled in the alluvial deposits along the dam centreline and upstream of the dam. The objectives of this work are:

   - To identify the quality and quantity of granular materials for use in embankment construction,
   - to locate and test suitable sources of concrete aggregate,
   - to establish the depth to bedrock in the valley bottom, and depth to impervious strata in the abutment areas along the centreline of the proposed cutoff, as well

as to establish potential difficulties in making the excavation for the cutoff,

- to carry out insitu permeability tests in overburden soils in the abutment areas,

- to establish the depth to bedrock along the centreline of the proposed low level conduit, as well as to established potential construction difficulties,

- to delineate the potential presence of and stratigraphy in the buried Tee Pee Valley under the west abutment area, and

- to establish the depth to bedrock along the centreline of the proposed toe drain.

2) High pressure injection tests should be undertaken in the bedrock, at the east abutment, along the dam centreline and near the toe of the embankment, in order to assess the stratigraphy and permeability of the bedrock at these locations.

3) Once the location and depth of the spillway has been confirmed, additional test holes should be drilled along the proposed centreline. The purpose of these holes is:

   - To confirm subsurface conditions along the spillway centreline and in particular to establish the volume of cemented sandstone which must be blasted,

   - to confirm groundwater conditions along the spillway centreline, and

   - to sample and test the shale bedrock in order to assess the potential use of this material in embankment construction.

4) Additional test pits and test holes should be drilled in potential borrow areas for clay till. Specificially, it is necessary to determine the following:

   - The susceptibility of the clay till to internal piping (dispersion tests are recommended),

- the water soluble sulphate content of the clay borrow, and

- the compaction properties of the clay borrow.

5) It is recommended that pneumatic piezometers be installed at selected locations in the vicinity of the damsite in order to establish current piezometric pressures in the bedrock at the site. This information is required in order to evaluate the stability of the proposed embankment.

6) A program of laboratory strength testing should be undertaken on samples of compacted clay till, on undisturbed alluvium and on undisturbed samples of bedrock core in order to estblish the effective stress strength and pore pressure parameters of these materials for use in stability analyses.

7) A suitable source of rip rap for upstream slope protection should be located.

8) An earth materials balance study should be carried out in order to optimize the configuration of the dam embankment as well as to establish the most efficient sequence of construction.

9) Selective stability and settlement analyses should be undertaken in order to refine the design of the embankment, particularly with regard to the presence of pre-existing failure planes within the interglacial (lower) alluvium on the west abutment.

10) Three deep seated landslides have been noted along the valley walls upstream from the dam embankment. Detailed geotechnical investigations will be required to assess the effects of reservoir impoundment on the stability of these landslides. This work should include an assessment of possible wave generation by landslide activity, and the risk of such a wave overtopping the dam embankment. The failure mechanism for the landslides should also be established, as this information will have an impact on the soil parameters used for the design of the dam embankment.

11) Detailed field checking of the valley slopes along the reservoir should be undertaken in order to identify other localized areas where the potential for instability is of concern.

12) A detailed hydrogeological assessment of the damsite and reservoir should be undertaken in order to assess the impact of the proposed reservoir on the groundwater regime. This work should include a more detailed evaluation of lateral seepage from the reservoir.

It is expected that 3 to 6 months would be required to complete the geotechnical investigations outlined above.

It is recommended that some phases of these geotechnical investigations be carried out at the pre-design phase. These should include:

1) Drilling and test pitting to delineate depth to bedrock at the proposed centreline and to establish potential construction difficulties with excavations for the cutoff and low level conduit,

2) Installation of a number of pneumatic piezometers in the bedrock to obtain a data base on piezometric levels for embankment stability analyses,

3) Drilling to delineate the potential presence of and stratigraphy in the buried Tee Pee Valley, and,

4) Drilling and analysis to establish the failure mechanism of three deep seated landslides along the valley walls upstream from the dam embankment.

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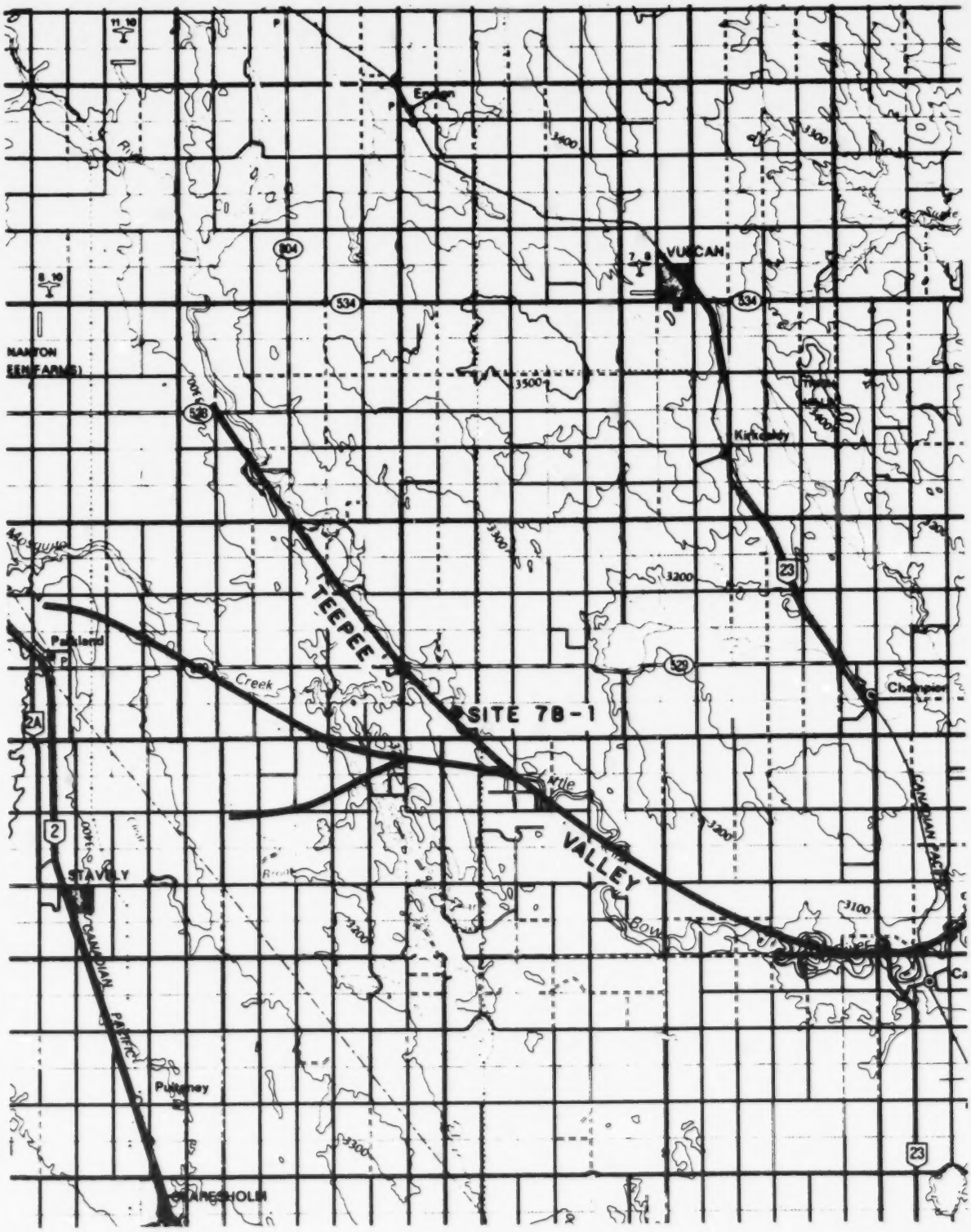
APPENDIX A

DRAWINGS

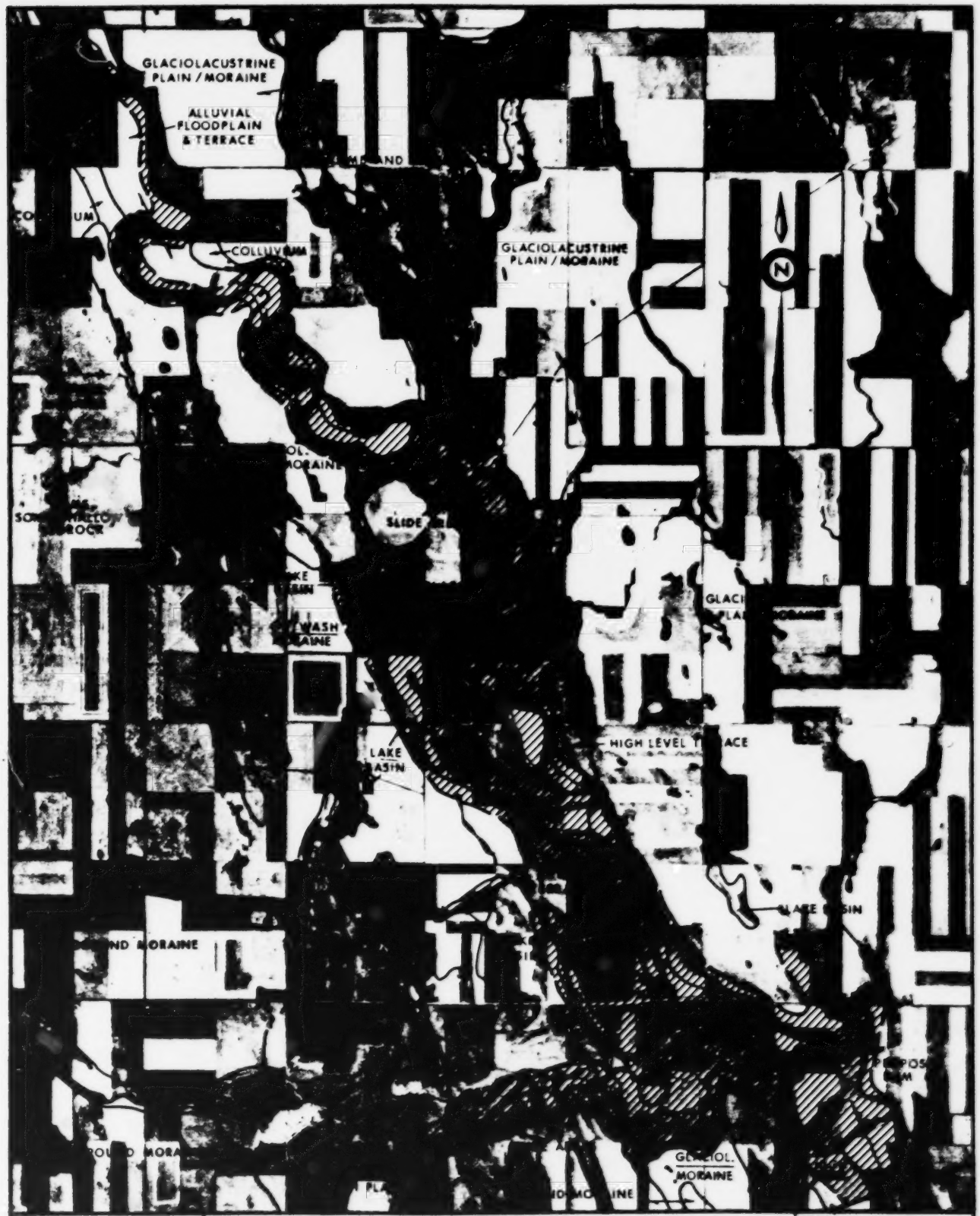


W-E-R ENGINEERING LTD.

LOCATION PLAN
PROPOSED DAM SITE 7B-1
LITTLE BOW RIVER BASIN

THURBER CONSULTANTS LTD., Geotechnical Engineers

DRAWN	IGJ / JAB
FILE NO.	19-293-1
DATE	SEPT. 1985
APPROVED	
SCALE	1:250,000
DRAWING NO	1



W-E-R ENGINEERING LTD.

SURFICIAL GEOLOGY
86,300 dam³ RESERVOIR

LITTLE BOW SITE 78-1

THURBER CONSULTANTS LTD., Geotechnical Engineers

DRAWN	IGJ / JAB
FILE NO.	19-293-1
DATE	SEPT. 1985
APPROVED	
SCALE	1:60,000
DRAWING NO	2

	W-E-R ENGINEERING LTD.	DRAWN: IGJ / JAB
	SURFICIAL GEOLOGY DAMSITE	FILE NO. 19-293-1
		DATE: SEPT. 1985
		APPROVED:
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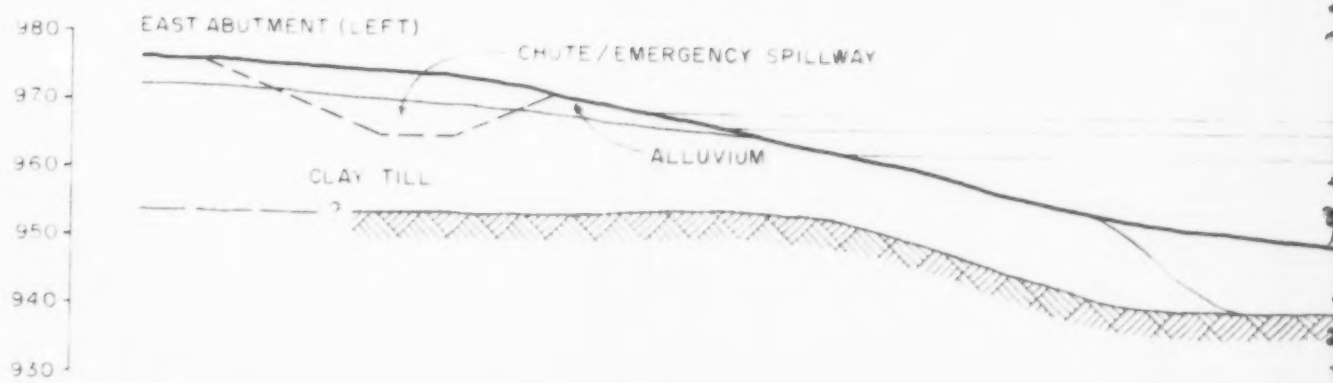
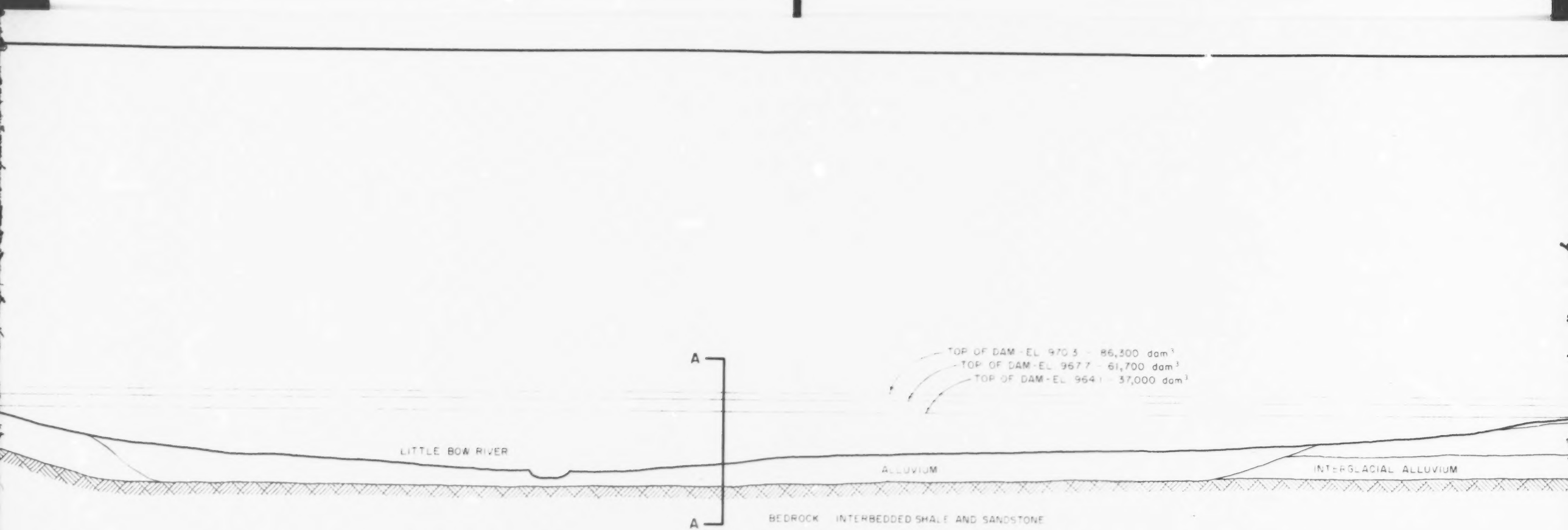
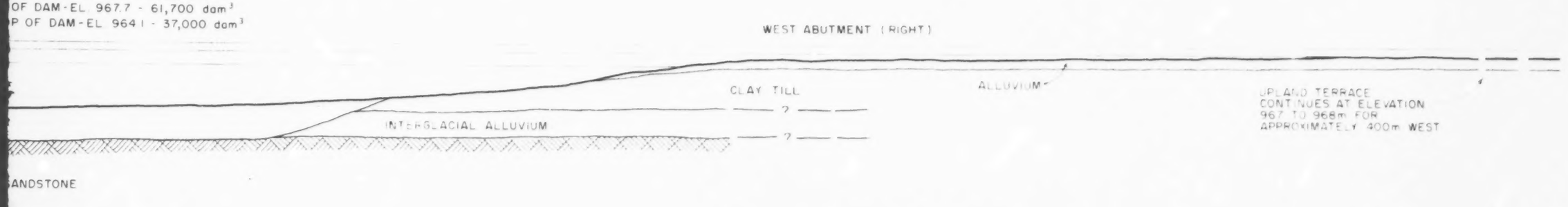

980
970
960
950
940
930
EAST ABUTMENT (LEFT)
CHUTE / EMERGENCY SPILLWAY
ALLUVIUM
CLAY TILL
?


A
A
TOP OF DAM - EL 970.3 - 86,300 dam³
TOP OF DAM - EL 967.7 - 61,700 dam³
TOP OF DAM - EL 964.1 - 37,000 dam³
LITTLE BOW RIVER
ALLUVIUM
INTERGLACIAL ALLUVIUM
BEDROCK - INTERBEDDED SHALE AND SANDSTONE

NOTES: 1. CROSS-SECTION PREPARED FROM INFORMATION PRESENTED IN PFRA REPORT ENTITLED "BOW RIVER DIVERSION TO OLDMAN RIVER (COMPONENT 928/8-27) APPENDIX B DIVERSION WORKS" APRIL 1971.

2. REFER TO DRAWINGS 5, 6 AND 7 FOR CROSS-SECTION A-A.

DAM - EL. 970.3 - 86,300 dam³
OF DAM - EL. 967.7 - 61,700 dam³
P OF DAM - EL. 964.1 - 37,000 dam³



SANDSTONE

RED FROM INFORMATION PRESENTED IN
LED" BOW RIVER DIVERSION TO OLDMAN RIVER
7) APPENDIX B DIVERSION WORKS" APRIL 1971.

5, 6 AND 7 FOR CROSS-SECTION A-A.

W-E-R ENGINEERING LTD.

GEOLOGICAL CROSS-SECTION

LITTLE BOW - SITE 7B-1

THURBER CONSULTANTS LTD., Geotechnical Engineers

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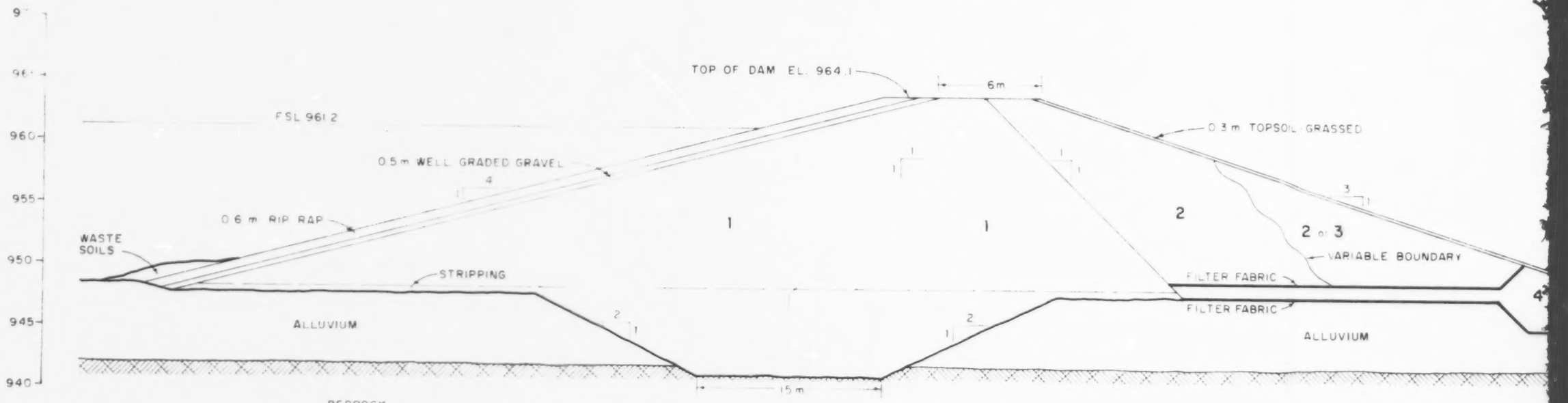


LEGEND

1 IMPERVIOUS MATERIAL, WELL COMPACTED CLAY TILL FROM THE UPLANDS.

2 PERVIOUS TO SEMI-PERVIOUS MATERIAL, RANDOM SANDS AND GRAVELS FROM CUTOFF EXCAVATION AND BORROW AREAS IN ALLUVIUM.

3 PERVIOUS TO SEMI-PERVIOUS MATERIAL, RECOMPACTED SHALE OR SANDSTONE BEDROCK IF AVAILABLE FROM SPILLWAY EXCAVATION.

4 PERVIOUS DRAIN, CLEAN SAND AND GRAVEL WITH LESS THAN 3% PASSING THE NO. 8 SIEVE.

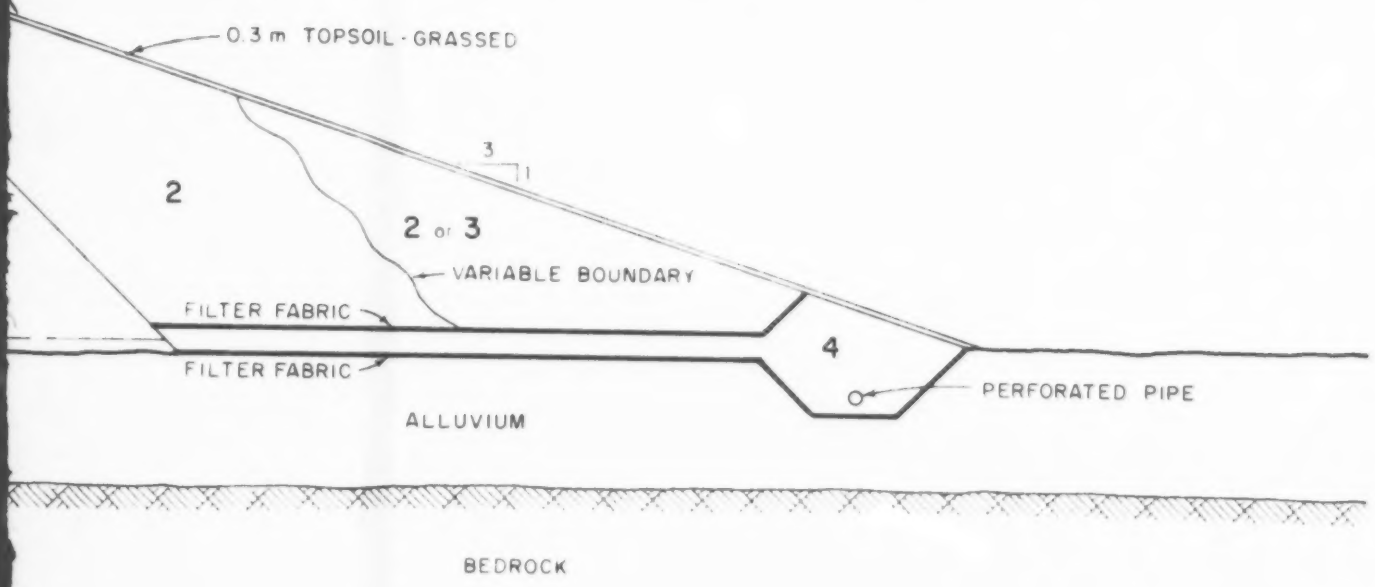
W-E-R E

PRELIMINARY E

37,000 d

LITTLE BOW - SITE 78-1

THURBER CONSULTANTS LTD.



W - E - R ENGINEERING LTD.

PRELIMINARY EMBANKMENT DESIGN

37,000 dam^3 CAPACITY

LITTLE BOW - SITE 7B - 1

THURBER CONSULTANTS LTD., Geotechnical Engineers

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DATE	SEPT. 1985
APPROVED	
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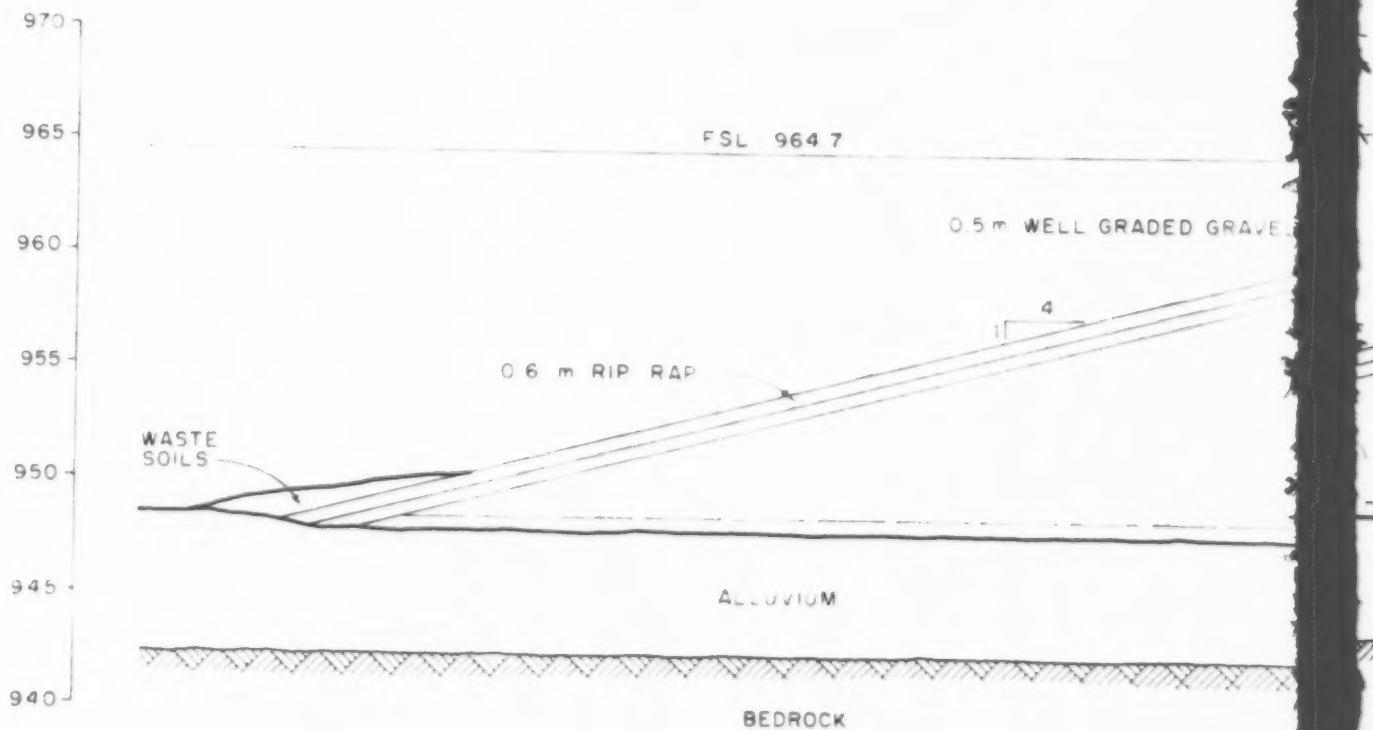
DRAWING NO.	5



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2 PERVIOUS TO S
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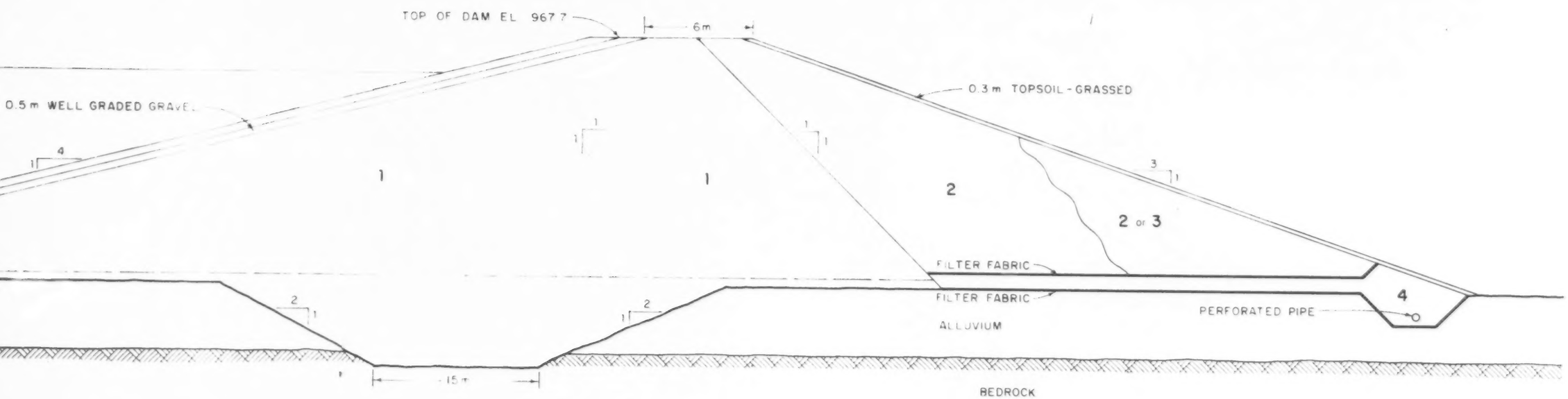

LEGEND

1 IMPERVIOUS MATERIAL, WELL COMPACTED CLAY TILL FROM THE UPLANDS.

2 PERVIOUS TO SEMI-PERVIOUS MATERIAL, RANDOM SANDS AND GRAVELS FROM CUTOFF EXCAVATION AND BORROW AREAS IN ALLUVIUM.

3 PERVIOUS TO SEMI-PERVIOUS MATERIAL, RECOMPACTED SHALE OR SANDSTONE BEDROCK IF AVAILABLE FROM SPILLWAY EXCAVATION.

4 PERVIOUS DRAIN, CLEAN SAND AND GRAVEL WITH LESS THAN 3% PASSING THE NO. 8 SIEVE.

W-E-R ENGINEERING LTD.

PRELIMINARY EMBANKMENT DESIGN

61,700 dam^3 CAPACITY

LITTLE BOW - SITE 78-1

THURBER CONSULTANTS LTD., Geotechnical Engineers

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DATE	SEPT. 1985
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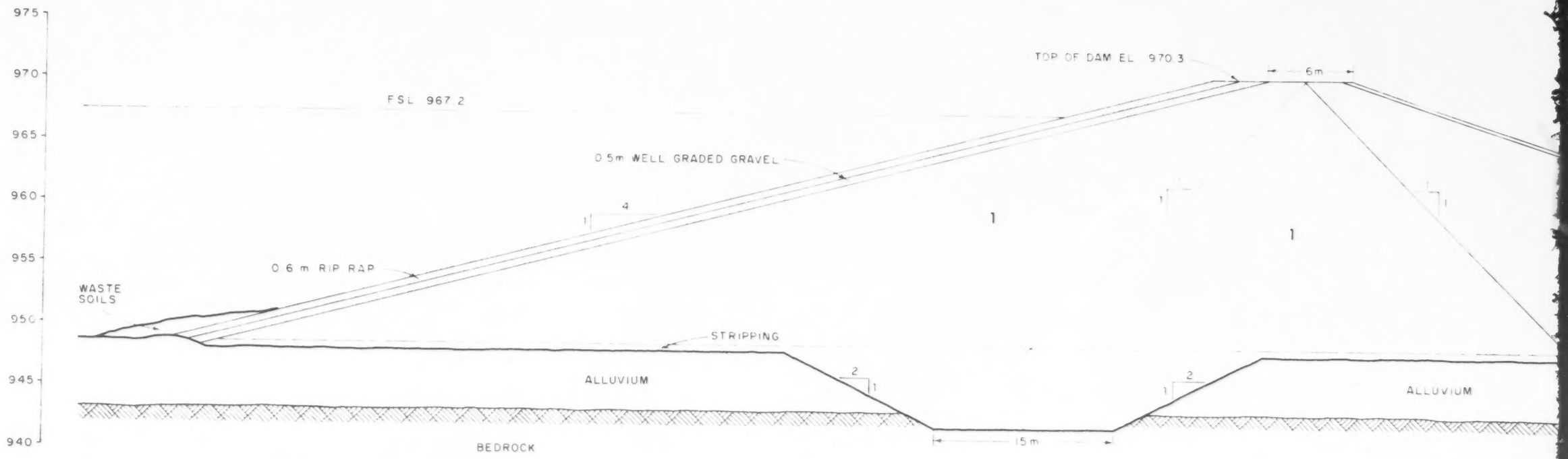


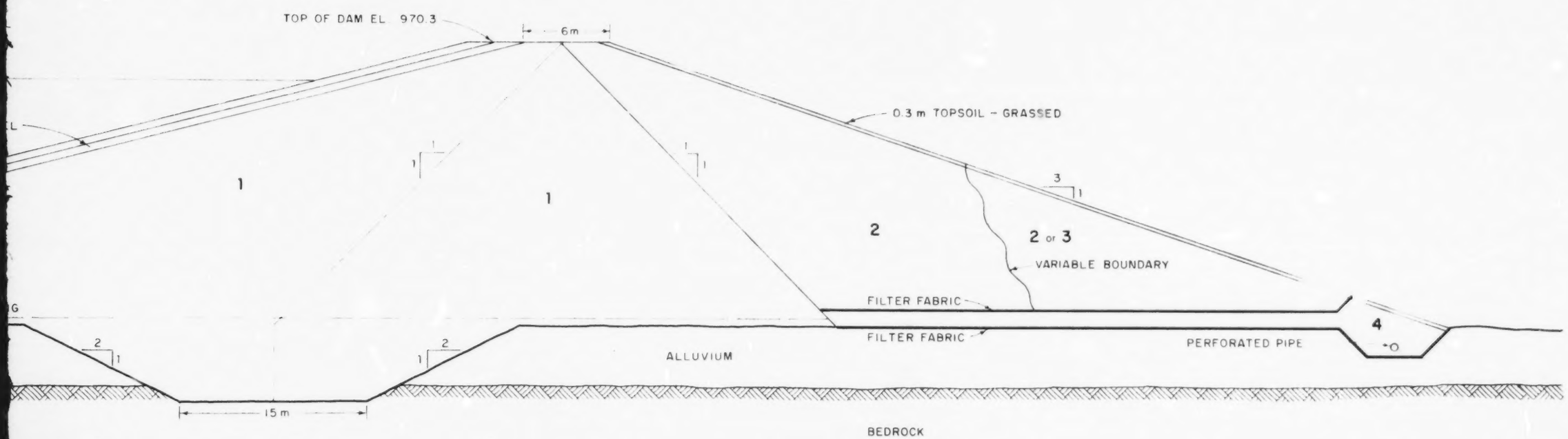
LEGEND

1 IMPERVIOUS MATERIAL, WELL COMPACTED CLAY TILL FROM THE UPLANDS.

2 PERVIOUS TO SEMI-PERVIOUS MATERIAL, RANDOM SANDS AND GRAVELS FROM CUTOFF EXCAVATION AND BORROW AREAS IN ALLUVIUM.

3 PERVIOUS TO SEMI-PERVIOUS MATERIAL, RECOMPACTED SHALE OR SANDSTONE BEDROCK IF AVAILABLE FROM SPILLWAY EXCAVATION.

4 PERVIOUS DRAIN, CLEAN SAND AND GRAVEL WITH LESS THAN 3% PASSING THE NO. 8 SIEVE.



LEGEND

ERIAL, WELL COMPACTED CLAY TILL FROM THE UPLANDS.

MI-PERVIOUS MATERIAL, RANDOM SANDS AND GRAVELS
XCAVATION AND BORROW AREAS IN ALLUVIUM.

MI-PERVIOUS MATERIAL, RECOMPACTED SHALE OR
ROCK IF AVAILABLE FROM SPILLWAY EXCAVATION.

CLEAN SAND AND GRAVEL WITH LESS THAN 3%
. 8 SIEVE.

W-E-R ENGINEERING LTD.

PRELIMINARY EMBANKMENT DESIGN

86,300 dam^3 CAPACITY

LITTLE BOW - SITE 78-1

THURBER CONSULTANTS LTD., Geotechnical Engineers

DRAWN	IGJ / JAB
FILE NO.	19-293-1
DATE	SEPT. 1985
APPROVED	
SCALE	1:300
DRAWING NO	7

LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
CONFLUENCE SITE (7B-1)
FEASIBILITY DESIGN REPORT
September, 1985

APPENDIX C

DETAILED SUMMARY OF DESIGN STUDIES

AND COST ESTIMATES

LIST OF TABLES

Title	Table No.
Detailed Cost Estimate - Little Bow Site 7B-1 37 000 dam^3 (30,000 ac.ft.)	C1
Pertinent Data - Little Bow Site 7B-1 37 000 dam^3 (30,000 ac.ft.)	C2
Detailed Cost Estimate - Little Bow Site 7B-1 61 700 dam^3 (50,000 ac.ft.)	C3
Pertinent Data - Little Bow Site 7B-1 61 700 dam^3 (50,000 ac.ft.)	C4
Detailed Cost Estimate - Little Bow Site 7B-1 86 300 dam^3 (70,000 ac.ft.)	C5
Pertinent Data - Little Bow Site 7B-1 86 300 dam^3 (70,000 ac.ft.)	C6

LIST OF DRAWINGS

Title	Drawing No.
Little Bow Site 7B-1 37 000 dam^3 (30,000 ac.ft.)	C1
Little Bow Site 7B-1 - 61 700 dam^3 (50,000 ac.ft.)	C2
Little Bow Site 7B-1 - 86 300 dam^3 (70,000 ac.ft.)	C3
Site 7B-1 Potential Highline Canal Locations	C4

DESIGN OF MAIN WORKS

1.0 General

This section covers the site layouts, hydraulic and structural design, and costing aspects of the main dam and appurtenant works together with potential highline canal locations for each of the reservoir storage alternatives under consideration for Little Bow Site 7B-1.

2.0 Base Data

These feasibility studies have utilized the hydrologic and storage data as provided in Appendix A, and the geotechnical assessment as outlined in Appendix B, in support of designs for the dams and structures. The report "Bow River Diversion to Oldman River - April 1971" by PFRA (Reference 1) was an important source of information for the present study.

Photogrammetric mapping for Site 7B-1, produced for the conceptual design was also used in this study. This mapping was prepared at a scale of 1:10,000 with two-metre contour intervals. In fringe areas of the reservoir this topography was complemented with the use of 1:50,000 NTS mapping.

3.0 Basic Project Requirements

The following subsections outline the project requirements established as a basis for preparing the feasibility designs and cost estimates for each of the three alternatives.

3.1 Reservoir Sizing

In accordance with the terms of this study, the reservoir alternatives were sized at 37 700, 61 700 and 86 300 dam^3 (30,000, 50,000 and 70,000 ac.ft.).

Based on topography and reservoir damage aspects it has been established that the physical maximum reservoir size at Site 78-1 would be about 86 300 dam^3 (70,000 ac.ft.). The area-capacity curve (see Section 2.0) used to establish FSL's for the three reservoir alternatives was essentially the same as that produced for the conceptual design study (Reference 2).

The relationship between available water supply and reservoir size is presented in the yield-storage curves provided in Appendix A.

3.2 Freeboard

Freeboard above the full supply level (FSL) for each dam alternative was calculated to include an allowance for wind setup and wave runup as well as appropriate allowances for gate operational failure and frost action.

3.3 Design Floods

Guidelines as established by the Dam Safety Branch of Alberta Environment were adhered to in setting the design criteria outlined below:

- spillways were sized to efficiently handle the 1:100 (structure design) flood routed outflow with a minimum of wave height freeboard on the dam.

- spillways are also capable of passing the 0.5 PMF (project flood) routed outflow with a minimum of 0.3 m of freeboard remaining on the dam. Hydrographs for the 1:100 and 0.5 PMF flood inflows are provided in Appendix A.

3.4 Low Level Irrigation Requirements/Storage Utilization

Requirements for the project dictate that the low level conduit be capable of releasing a flow of 8.5 m^3/s (300 cfs) at minimum reservoir levels, for irrigation purposes. The sill level of the conduit intake structure was set slightly below the floodplain, for each alternative, in order to fully utilize the reservoir's storage capability, without encroaching on the live storage.

4.0 General Layout and Selection of Elements

4.1 General

For the 86 300 dam^3 (70,000 ac.ft.) reservoir alternative the dam centreline location proposed by PFRA in 1971 (Reference 1) was retained. However, the embankment centrelines for the 61 700 dam^3 (50,000 ac.ft.) and 37 000 dam^3 (30,000 ac.ft.) alternatives were located approximately 200 m upstream of the PFRA site as in the case of the conceptual design study (Reference 2). This was done so that the east gully could form part of the spillway approach channel to reduce excavation quantities. Embankment designs are based on the geotechnical investigations presented in this report.

In all other respects, the layouts were determined on the basis of topographic suitability. The original PFRA spillway location was essentially retained for all three storage alternatives.

The conduit location selected by PFRA was also judged as most suitable in terms of foundation and topography. Potential highline canal locations have also been incorporated into the structure layouts.

The general layouts as shown in Figures C1, C2 and C3 represent the feasibility level structural investigations as carried out for this study. Highline canal locations are shown in Figure C4.

Embankment designs used in this investigation are outlined in Appendix B. The embankments are intended to utilize materials readily available at the site. The cost estimates presented herein reflect the nature of the material excavated and the quantities of borrow required.

The proposed embankment sections are zone-type earthfills with 4:1 upstream and 3:1 downstream slopes and a 6 m crest width. Seepage under the dam would be controlled by a cutoff trench extending through the pervious alluvium to impervious bedrock or glacial till. A granular filter in conjunction with a toe drain will be required to control seepage through the embankment. Slope protection will consist of rock riprap placed on a gravel bed.

The predicted embankment settlement at the damsite for each alternative requires that crests be cambered to ensure maintenance of the required freeboard should such movements occur. However, dam crest elevations shown in the structure layouts only reflect the levels required to accommodate the selected reservoir sizes. Precise elevations along the crest (including camber) would be determined during final

design. None-the-less, embankment quantities calculated for costing purposes include an allowance for this configuration.

4.2 Spillways

a) Hydraulic Design

The spillways proposed for each alternative are gated chute type structures with hydraulic jump stilling basins. Spillways equipped with radial gates were selected as being the most cost effective. It has been assumed that gate operators will be available for the project, however, normal gate operation will be automated.

Spillway dimensions are based on optimization studies to determine the most economical combination of crest length and gate height to pass the required flood flows. Chutes are flared into the stilling basin to ensure that flow energy is adequately dissipated.

The spillways are designed to efficiently pass the routed 1:100 year flood (structure design flood). The structures are also sized to handle floods up to one-half the probable maximum flood (0.5 PMF) with the possibility of limited damage being sustained in the lower structural components of the spillway during the passage of such floods.

The sizing of the radial gates and determination of the corresponding routed outflows is dependent on assumptions regarding the operation of the gates during flood events. For the purposes of this report it was assumed that during flood events the gates would be operated automatically to

maintain the reservoir level at FSL as long as possible and then to completely open when the reservoir level rises above FSL. The rating curves provided are based on an average weir coefficient of 2.10 (metric) with the gates fully opened during passage of floods.

Entrance lengths for the control structure have been increased beyond hydraulic requirements to allow for piers, radial gates, hoists and a service bridge deck.

Basic spillway dimensions for each site may be found in Figures C1, C2 and C3, and Tables C2, C4 and C6.

b) Location

Spillway structures have been generally located to take advantage of topographic and foundation conditions. Spillway profiles are configured to minimize excavation depths, although all three spillways would intercept bedrock layers and some degree of rebound is anticipated. Approach channels, which originate in the gully on the east abutment, have been aligned to ensure that high velocities do not endanger the upstream abutment of the dam. Avoidance of downstream erosion near the toe of the dam was the main objective in the exit channel alignment. Materials excavated from the spillways can be utilized in the main dam embankments.

c) Structural Design

The spillway structures are of conventional reinforced concrete design, with structural components sized approximately by comparison with similar existing works. The required space and supports for the radial gates, hoisting

systems and upstream bulkheads are provided by adequately sized piers and abutments. Gate hoists, controls and additional equipment would be located on a hoisting deck spanning between the piers. The deck would also act as an access bridge across the spillway to accommodate service vehicles and equipment.

Conventional jointing, cutoffs and key walls have been incorporated into the designs to allow for articulation should differential foundation movement occur. The proposed chute and stilling basin sections consist of cantilever side walls and interior floor slabs articulated from the walls. In the lower chute and basin areas, concrete thicknesses have been increased to withstand the large dynamic forces and ensure stability.

d) Foundation and Drainage

Upstream and downstream cutoff walls and intermediate cross-walls are provided to control foundation seepage.

Where foundations in the region of the crest section are fissured or pervious, some seepage control will be necessary through drainage conduits and relief wells. For the chute and stilling basin sections a granular bedding layer is proposed for the purposes of seepage and uplift pressure control. In the chute area, seepage through the foundation will be relieved by means of pipe drains located directly under the slab with gravity outlets to drainage conduits located parallel to and outside the retaining walls. In the basin area where substantially greater hydrostatic uplift forces are possible a curtain wall and pressure relief pipes connected to the subfloor drainage system are proposed to control uplift pressures.

4.3 Low Level Irrigation Conduits

Irrigation conduits would consist of precast concrete pipelines with a reinforced concrete inlet, gatewell and outlet structures. Flow in the conduit will be controlled by means of sluice gates installed in the gatewell. The gatewell, located just upstream of the dam centreline will be a two-celled structure equipped with both service and auxiliary gates complete with hoisting equipment.

Details of these conduits, including basic dimensions, are shown in Figures C1, C2, and C3, and Tables C2, C4 and C6.

To ensure maximized live storage, irrigation conduits have been designed for the required release of 300 cfs at minimum reservoir levels, assuming full pipe flow with outlet control. Outlet structures will be hydraulic jump basins designed to handle the full gate, maximum head condition. Gate shafts will be equipped with air supply piping to prevent gate cavitation. Conduit gradients have been set to minimize slug flow or other adverse hydraulic action.

Conduits have been located on the east abutment of the river valley, excavated to some depth into the floodplain material to ensure that structure foundation pads rest firmly on competent bedrock. To compensate for the relatively deep-set conduit a short drop inlet riser is proposed for intake purposes. During final design, consideration should be given to locating the conduit somewhat further to the east on bedrock. This may be desirable provided competent shallow bedrock can be located in that vicinity and the conduit outlet does not interfere with the spillway outlet.

Conduits will also be utilized for construction diversion purposes and be converted to permanent irrigation works at completion of the project.

4.4 Highline Canals

Preliminary layouts of highline irrigation canals are proposed for each of the three alternative storages under consideration (Drawing C4). These canal layouts are based on the assumption that 50 percent of the storage of alternative reservoir capacities will be available for diversion to a highline irrigation canal. The remaining 50 percent of storage would be used for lowline irrigation.

The proposed highline canal locations essentially follow 25 m contour lines available from 1:50,000 NTS mapping. The layouts would involve a certain amount of side hill canal for approximately 10 km, at which point the canals head out of the Little Bow River valley. This would be closely examined in a pre-design study.

Neither irrigation headworks nor canals were costed for this study.

Numerous other storage combinations can, of course, be considered. This would be part of final design studies upon implementation of Site 78-1.

5.0 Basis of Costing

Detailed cost estimates for the proposed works can be found in Tables C1, C3 and C5. These estimates are intended to include all costs associated with construction of the dam embankments and ancillary works, including contingencies.

Allowances for engineering, design and surveys have also been included. However, interest during construction and annual charges have been omitted.

Unit prices utilized in the preparation of cost estimates for this study are based on bid prices from recent tenders on similar storage projects, as well as current tender prices on major irrigation works, in Southern Alberta.

The intended scope and/or coverage of specific items which appear in the detailed cost estimate tables are as follows:

1. Care of Water - is assumed to cover the cost of miscellaneous works for dewatering the site area during construction, but does not include the cost of major items for river or creeks diversion such as conduits and cofferdams.

2. Excavations - are assumed to cover all costs necessary to excavate and haul material from its source to place of deposition. Costs include such work as stripping, trimming, ripping, etc.

3. Embankments - are assumed to cover all costs necessary to place and compact material in the embankment or backfill areas. Costs include allowances for mixing material, adding, moisture, etc.

4. Embankment Filter and Drainage System - is assumed to cover all costs necessary to supply and place downstream filter and drainage systems including perforated pipes and relief wells.

5. Rock Riprap - is assumed to cover all costs necessary to harvest, process, haul and place riprap.

6. Granular Blanket Below Riprap - is assumed to cover all costs necessary to excavate, process, haul and place materials on the embankment.

7. Shotcrete Abutment Treatment - is assumed to cover all costs necessary to prepare the abutment adjacent to the impervious clay core, including shotcreting.

8. Concrete Items - are assumed to cover all costs necessary to produce and place concrete as shown on the drawings. Costs include the supply and placement of reinforcement steel and formwork, processing of aggregates and finishing.

9. Spillway Drainage System - is assumed to include all costs associated with the subfloor drainage system under the spillway chute and stilling basin and the conduit outlet basin.

10. Gate Items - are assumed to cover all costs necessary to manufacture and install complete working gates including hoisting systems.

11. Miscellaneous Items - are assumed to cover items which are anticipated at this time as being required, but which are too small or too detailed to include in the cost table.

12. Contingencies - are assumed to cover items of unknown or extra work which cannot be anticipted at this time because of the preliminary nature of the study and the lack of detailed background or field information.

TABLE C1
DETAILED COST ESTIMATE
37 000 DAM3 (30,000 AC.FT.)

	Quantity	UNIT $	EXTENSION
CARE OF WATER	lump sum	SUBTOTAL	210,000
EMBANKMENT			
Foundation Preparation			
-Stripping	12,400 M3	1.65	20,460
-Key Trench Excavation	83,000 M3	2.00	166,000
Compacted Embankment			
- Zone #1	304,000 M3	0.90	273,600
- Zone #2	90,600 M3	0.90	81,540
Borrow Excavation	232,000 M3	2.00	464,000
Embankment Filter and Drainage System			
- Zone #4	13,800 M3	21.00	289,800
- Filter Cloth	25,250 M2	1.65	41,663
Rock Riprap	14,000 M3	35.00	490,000
Riprap Bedding	11,700 M3	16.00	187,200
Topsoil D/S Face Of Dam	5,500 M3	1.65	9,075
SUBTOTAL EMBANKMENT			2,023,000
SPILLWAY			
Excavation and Haul			
- Rock	6,400 M3	6.50	41,600
- Other	242,100 M3	2.25	544,725
Zone #1 Backfill	5,500 M3	3.90	21,450
Reinforced Concrete	1,596 M3	525.00	837,900
Drainage System	750 M3	25.00	18,750
Radial Gates & Hoists(3 sets)	lump sum		90,000
Miscellaneous Items			77,721
SUBTOTAL SPILLWAY			1,632,000
IRRIGATION CONDUIT			
Excavation - Structural	10,800 M3	2.65	28,620
Inlet and Outlet Channel Excavation	5,000 M3	2.65	13,250
Zone #1 Backfill	11,800 M3	3.90	46,020
Reinforced Concrete Pipe	115 M	1,250.00	143,750
Structural Concrete	285 M3	785.00	223,725
Control Gates & Hoists(2 sets)	lump sum		60,000
Miscellaneous Items			25,768
SUBTOTAL CONDUIT			541,000
SUBTOTAL CONSTRUCTION			$4,406,000
Contingencies(20%)			$881,000
TOTAL CONSTRUCTION COST			$5,288,000

TABLE C2
Pertinent Data
Little Bow Site 78-1
37 000 dam^3 (30,000 ac.ft.) Storage Alternative

Reservoir

Full Supply Level (FSL)	961.2 m
Capacity at FSL	37 000 dam^3
Live Storage	36 000 dam^3
Flooded Area at FSL	575 ha
Maximum Depth at FSL	16.1 m
Project Design Flood - 0.5 PMF (inflow)	322.8 m^3/s
Spillway Design Flood - 1:100 (inflow)	147.3 m^3/s
Maximum Water Level	963.8 m

Embankment

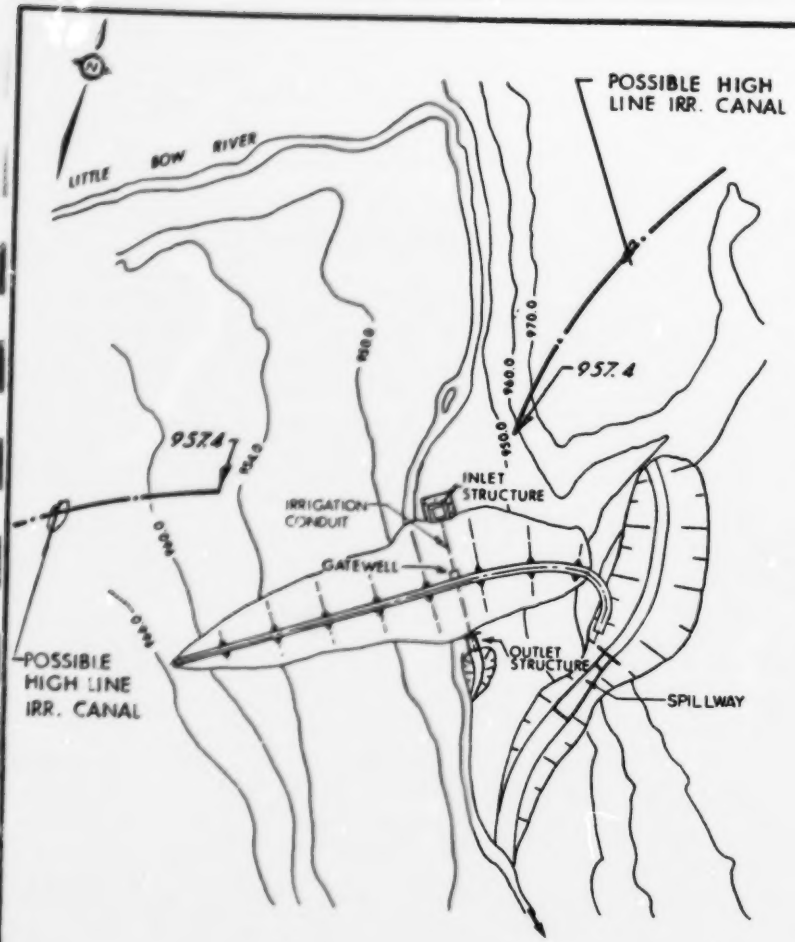
Top of Dam Elevation	964.1 m
Crest Length	442 m
Maximum Height	19.0 m
Volume of Fill	394 600 m^3
Freeboard at Spillway Design Flood	2.5 m
Freeboard at Project Design Flood	0.3 m

Spillway

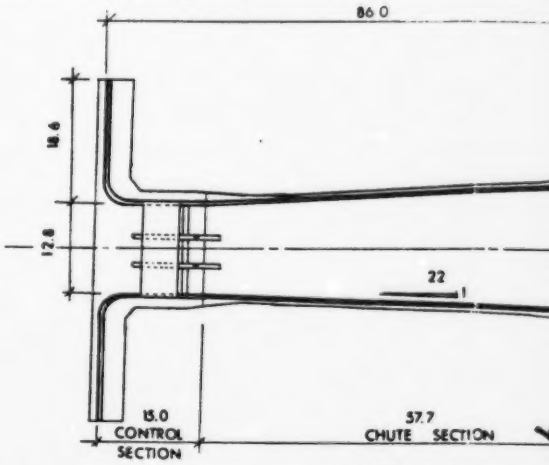
Crest Elevation	959.1 m
Radial Gates	3 @ 4.0 m (W) x 2.1 m (H)
Net Weir Length	12.0 m
Gross Crest Width	12.8 m
Chute Width	varies 12.8 m to 18.0 m
Basin Width	18.0 m
Structure Length	86.0 m
Basin Elevation	943.3 m
Design Discharge (1:100 routed outflow)	99.6 m^3/s
Probable Maximum Discharge (0.5 PMF routed outflow)	265.0 m^3/s

Irrigation Supply Conduit

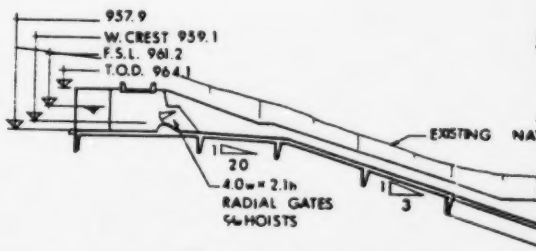
Conduit Diameter	1.5 m
Conduit Length	115 m
Irrigation Supply Capacity Requirement	8.5 m^3/s
Gate Size	2 @ 2 m x 2 m
Intake Crest Elevation	949.0 m
Basin Elevation	944.0 m
Minimum Reservoir Elevation to Pass Irrigation Requirement	949.7 m



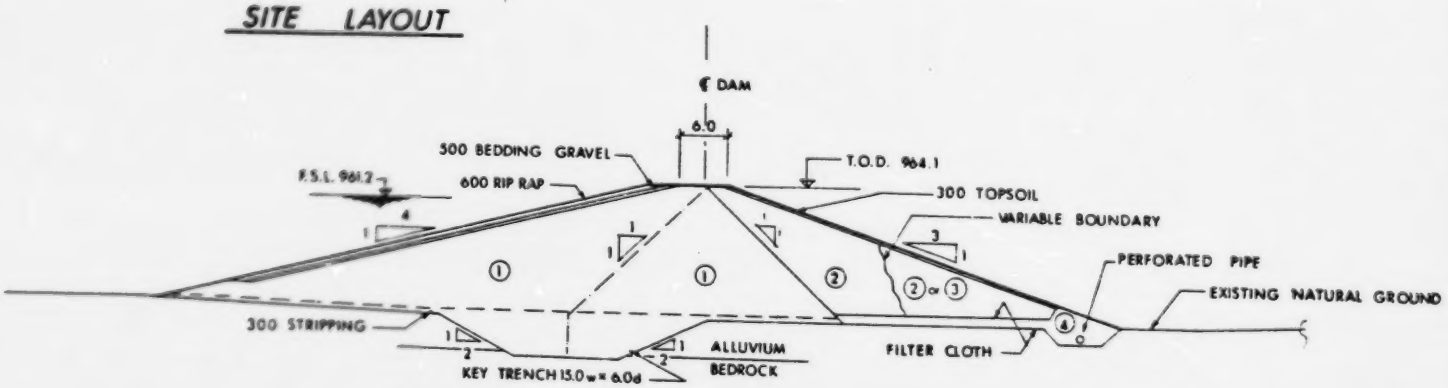
SITE LAYOUT



SPILLWAY PLA

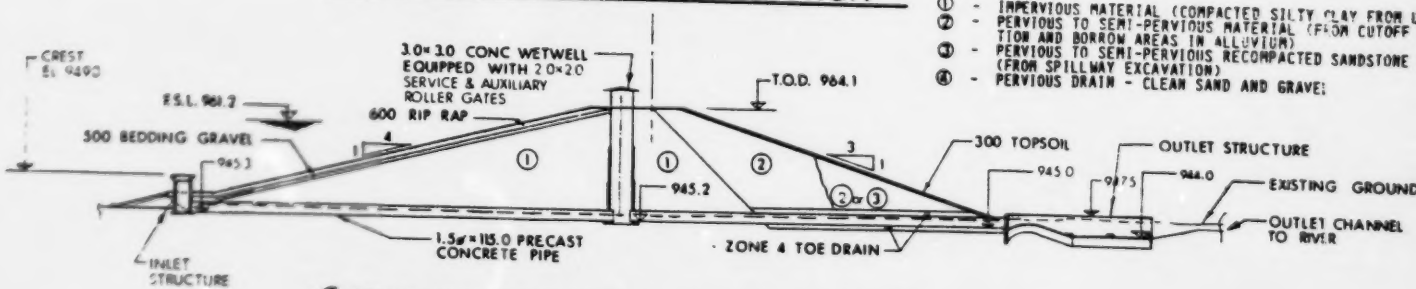


℄ SECTION - SPILLWAY

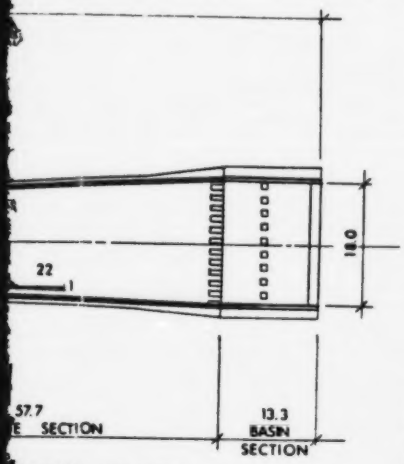


MAXIMUM EMBANKMENT SECTION

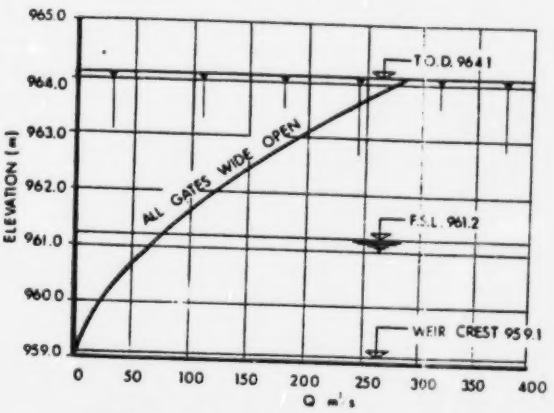
① - IMPERVIOUS MATERIAL (COMPACTED SILTY CLAY FROM L
② - PERVIOUS TO SEMI-PERVIOUS MATERIAL (FROM CUTOFF TION AND BORROW AREAS IN ALLUVIUM)
③ - PERVIOUS TO SEMI-PERVIOUS RECOMPACTED SANDSTONE (FROM SPILLWAY EXCAVATION)
④ - PERVIOUS DRAIN - CLEAN SAND AND GRAVEL



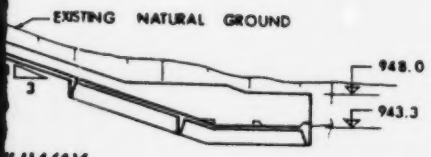
℄ SECTION - IRRIGATION SUPPLY CONDUIT AND DIVERSION WORKS



LWAY PLAN



SPILLWAY RATING CURVE



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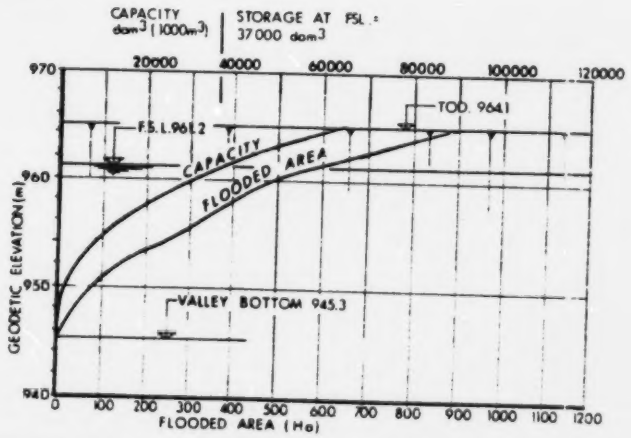


ATURAL GROUND

FLOODED AREA AND CAPACITY CURVES

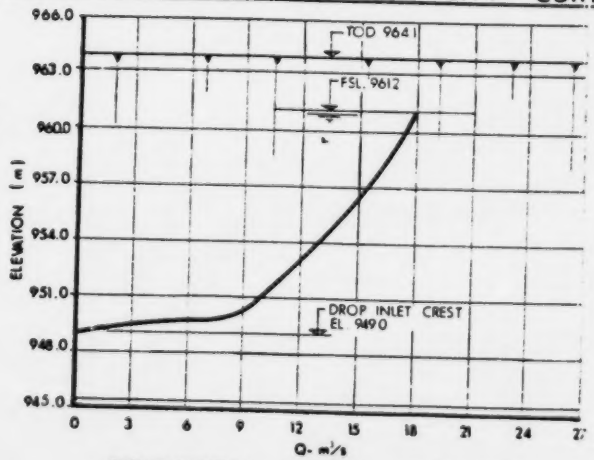
SILTY CLAY FROM UPLANDS)
RIAL (FROM CUTOFF EXCAVA-
IUM)
MPACTED SANDSTONE + SHALE

D GRAVEL

CTURE

EXISTING GROUND

OUTLET CHANNEL
TO RIVER



IRRIGATION SUPPLY CONDUIT RATING CURVE

Notes:

REV	DATE	REVISION DESCRIPTION	ENG	APPR.

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT
LITTLE BOW RIVER BASIN
STORAGE INVESTIGATION
CONFLUENCE SITE 7B-1
FEASIBILITY DESIGNS

37000 dam³ STORAGE ALTERNATIVE
(30,000 ac. ft)

GENERAL LAYOUT

DESIGNED	CHECKED	DATE	JOB No	DRAWING No.
P.B.				C1
DRAFTED	APPROVED	SCALE	SHEET No	
E.O.				

TABLE C3
DETAILED COST ESTIMATE
61 700 DAM3 (50,000 AC.FT.)

	Quantity	UNIT $	EXTENSION
CARE OF WATER	lump sum	SUBTOTAL	242,000
EMBANKMENT			
Foundation Preparation			
-Stripping	17,800 M3	1.65	29,370
-Key Trench Excavation	88,000 M3	2.00	176,000
Compacted Embankment			
- Zone #1	431,000 M3	0.80	344,800
- Zone #2	143,000 M3	0.80	114,400
Borrow Excavation	517,000 M3	1.50	775,500
Embankment Filter and Drainage System			
- Zone #4	16,359 M3	21.00	343,539
- Filter Cloth	32,450 M2	1.65	53,543
Rock Riprap	19,300 M3	35.00	675,500
Riprap Bedding	16,100 M3	16.00	257,600
Topsoil D/S Face Of Dam	7,700 M3	1.65	12,705
SUBTOTAL EMBANKMENT			2,783,000
SPILLWAY			
Excavation and Haul			
- Rock	5,500 M3	6.50	35,750
- Other	145,300 M3	2.25	326,925
Zone #1 Backfill	4,700 M3	3.90	18,330
Reinforced Concrete	1,639 M3	525.00	860,475
Drainage System	810 M3	25.00	20,250
Radial Gates & Hoists(3 sets)	lump sum		99,000
Miscellaneous Items			68,037
SUBTOTAL SPILLWAY			1,429,000
IRRIGATION CONDUIT			
Excavation - Structural	12,600 M3	2.65	33,390
Inlet and Outlet Channel Excavation	5,000 M3	2.65	13,250
Zone #1 Backfill	13,600 M3	3.90	53,040
Reinforced Concrete Pipe	140 M	1,250.00	175,000
Structural Concrete	323 M3	785.00	253,555
Control Gates & Hoists(2 sets)	lump sum		70,000
Miscellaneous Items			29,912
SUBTOTAL CONDUIT			628,000
SUBTOTAL CONSTRUCTION			$5,082,000
Contingencies(20%)			$1,016,000
TOTAL CONSTRUCTION COST			$6,098,000

TABLE C4
Pertinent Data
Little Bow Site 7B-1
61 700 dam^3 (50,000 ac.ft.) Storage Alternative

Reservoir

Full Supply Level (FSL)	964.7 m
Capacity at FSL	61 700 dam^3
Live Storage	60 700 dam^3
Flooded Area at FSL	865 ha
Maximum Depth at FSL	19.7 m
Project Design Flood - 0.5 PMF (inflow)	322.8 m^3/s
Spillway Design Flood - 1:100 (inflow)	147.3 m^3/s
Maximum Water Level	967.4 m

Embankment

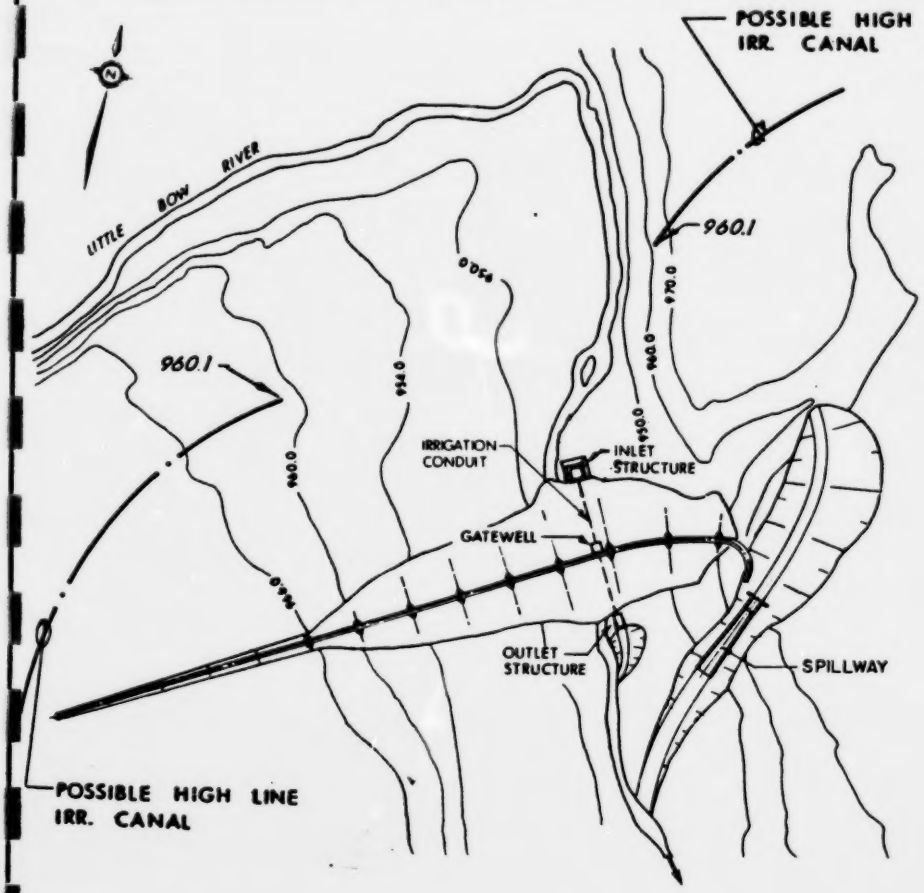
Top of Dam Elevation	967.4 m
Crest Length	810 m
Maximum Height	22.7 m
Volume of Fill	574 000 m^3
Freeboard at Spillway Design Flood	2.6 m
Freeboard at Project Design Flood	0.3 m

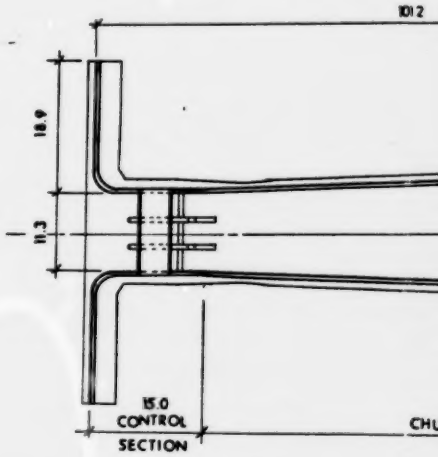
Spillway

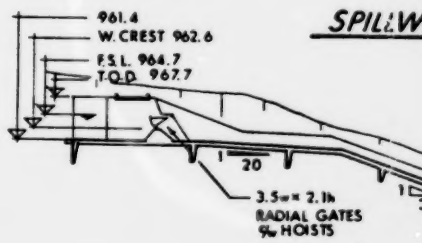
Crest Elevation	962.6 m
Radial Gates	3 @ 3.5 m (W) x 2.1 m (H)
Net Weir Length	10.5 m
Gross Crest Width	11.3 m
Chute Width	varies 11.3 m to 16.5 m
Basin Width	16.5 m
Structure Length	101.2 m
Basin Elevation	943.1 m
Design Discharge (1:100 routed outflow)	84.6 m^3/s
Probable Maximum Discharge (0.5 PMF routed outflow)	239.2 m^3/s

Irrigation Supply Conduit

Conduit Diameter	1.5 m
Conduit Length	140 m
Irrigation Supply Capacity Requirement	8.5 m^3/s
Gate Size	2 @ 2 m x 2 m
Intake Crest Elevation	949.0 m
Basin Elevation	944.0 m
Minimum Reservoir Elevation to Pass Irrigation Requirement	949.7 m



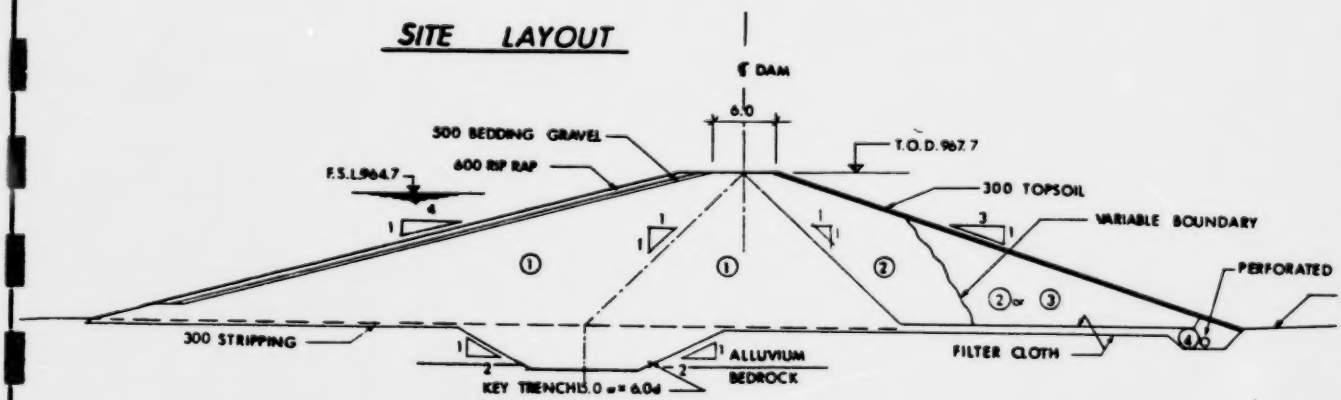




SPILLW

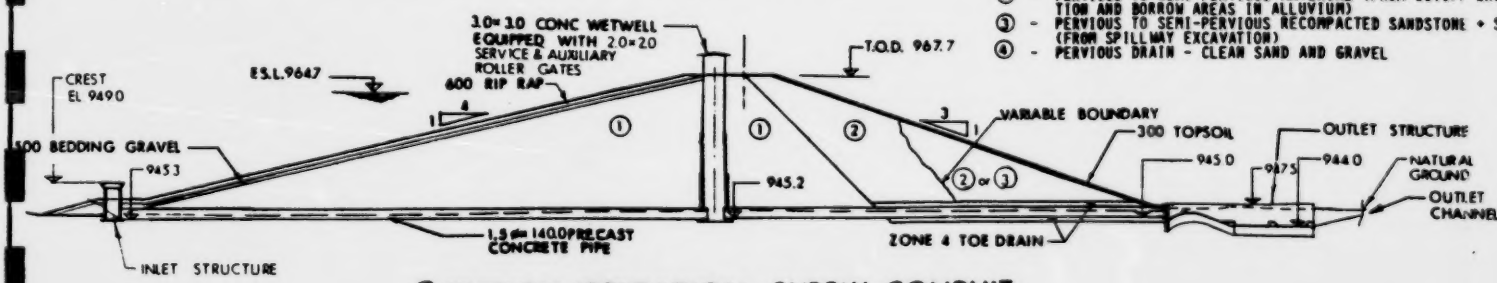
℄ SECTION - SPILL

SITE LAYOUT

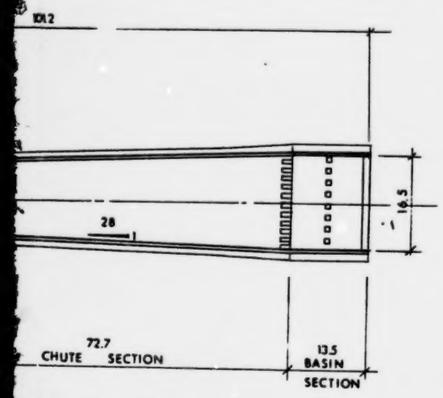


MAXIMUM EMBANKMENT SECTION

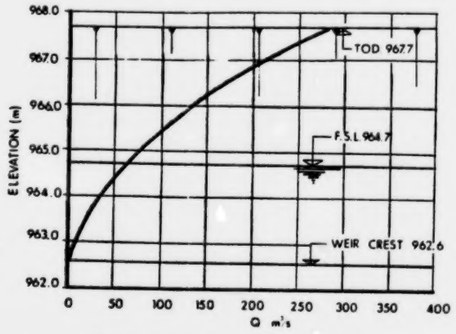
① - IMPERVIOUS MATERIAL (COMPACTED SILTY CLAY FROM UPLA
② - PERVIOUS TO SEMI-PERVIOUS MATERIAL (FROM CUTOFF EXC
TION AND BORROW AREAS IN ALLUVIUM)
③ - PERVIOUS TO SEMI-PERVIOUS RECOMPACTED SANDSTONE + S
(FROM SPILLWAY EXCAVATION)
④ - PERVIOUS DRAIN - CLEAN SAND AND GRAVEL



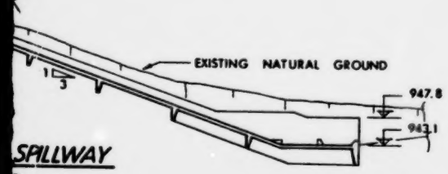
℄ SECTION - IRRIGATION SUPPLY CONDUIT AND DIVERSION WORKS



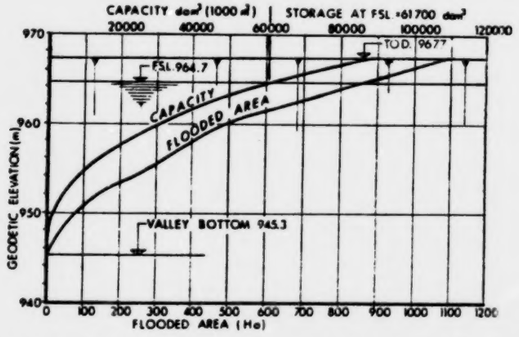
SPILLWAY PLAN



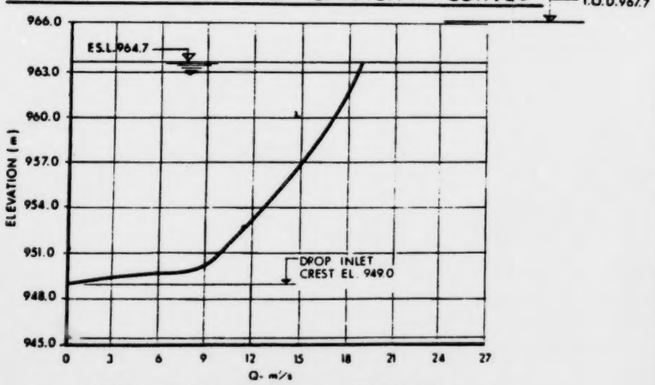
SPILLWAY RATING CURVE



SPILLWAY



FLOODED AREA AND CAPACITY CURVES



IRRIGATION SUPPLY CONDUIT RATING CURVE

REV	DATE	REVISION DESCRIPTION	ENG.	APPR.

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT: LITTLE BOW RIVER BASIN STORAGE INVESTIGATION CONFLUENCE SITE 7B-1 FEASIBILITY DESIGNS

61 700dam³ STORAGE ALTERNATIVE (50,000ac.ft)

GENERAL LAYOUT

DESIGNED	CHECKED	DATE	JOB No	DRAWING No
P.B.		SEP 16 81		C2
DRAFTED	APPROVED	SCALE	SHEET No	
E.O.		N.T.S.		

TABLE C5
DETAILED COST ESTIMATE
86300 DAM3 (70,000 AC.FT.)

	Quantity	UNIT $	EXTENSION
CARE OF WATER	lump sum	SUBTOTAL	295,000
EMBANKMENT			
Foundation Preparation			
-Stripping	24,200 M3	1.65	39,930
-Key Trench Excavation	128,000 M3	1.90	243,200
Compacted Embankment			
- Zone #1	604,000 M3	0.70	422,800
- Zone #2	196,000 M3	0.70	137,200
Borrow Excavation	815,000 M3	1.50	1,222,500
Embankment Filter and Drainage System			
- Zone #4	22,300 M3	21.00	468,300
- Filter Cloth	42,300 M2	1.65	69,795
Rock Riprap	26,000 M3	35.00	910,000
Riprap Bedding	21,600 M3	16.00	345,600
Topsoil D/S Face Of Dam	9,950 M3	1.65	16,418
SUBTOTAL EMBANKMENT			3,876,000
SPILLWAY			
Excavation and Haul			
- Rock	5,500 M3	6.50	35,750
- Other	95,400 M3	2.25	214,650
Zone #1 Backfill	4,550 M3	3.90	17,745
Reinforced Concrete	1,661 M3	525.00	872,025
Drainage System	920 M3	25.00	23,000
Radial Gates & Hoists(3 sets)	lump sum		108,000
Miscellaneous Items			63,559
SUBTOTAL SPILLWAY			1,335,000
IRRIGATION CONDUIT			
Excavation - Structural	14,200 M3	2.65	37,630
Inlet and Outlet Channel Excavation	5,000 M3	2.65	13,250
Zone #1 Backfill	15,200 M3	3.90	59,280
Reinforced Concrete Pipe	158 M	1,250.00	197,500
Structural Concrete	348 M3	785.00	273,180
Control Gates & Hoists(2 sets)	lump sum		80,000
Miscellaneous Items			33,042
SUBTOTAL CONDUIT			694,000
SUBTOTAL CONSTRUCTION			$6,200,000
Contingencies(20%)			$1,240,000
TOTAL CONSTRUCTION COST			$7,440,000

TABLE C6
Pertinent Data
Little Bow Site 7B-1
86 300 dam^3 (70,000 ac.ft.) Storage Alternative

Reservoir

Full Supply Level (FSL)	967.2 m
Capacity at FSL	86 300 dam^3
Live Storage	85 300 dam^3
Flooded Area at FSL	1,075 ha
Maximum Depth at FSL	21.9 m
Project Design Flood - 0.5 PMF (inflow)	322.8 m^3/s
Spillway Design Flood - 1:100 (inflow)	147.3 m^3/s
Maximum Water Level	970.0 m

Embankment

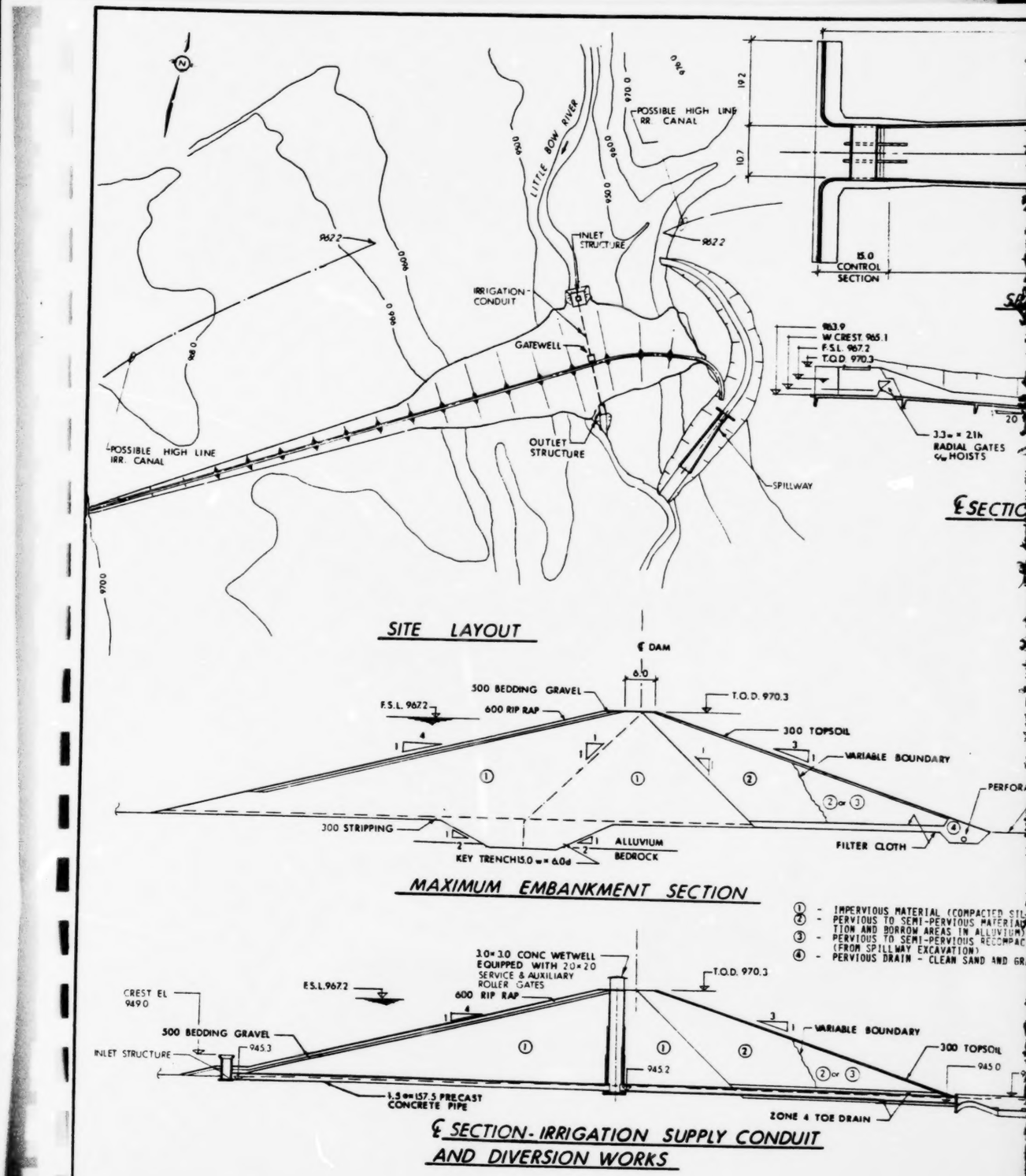
Top of Dam Elevation	970.3 m
Crest Length	1 180 m
Maximum Height	25.0 m
Volume of Fill	800 000 m^3
Freeboard at Spillway Design Flood	2.7 m
Freeboard at Project Design Flood	0.3 m

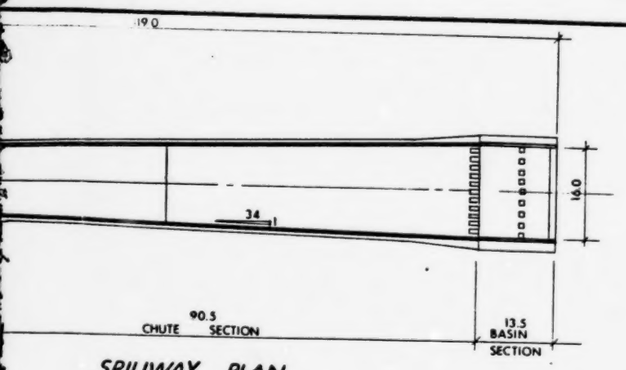
Spillway

Crest Elevation	965.1 m
Radial Gates	3 @ 3.3 m (W) x 2.1 m (H)
Net Weir Length	9.9 m
Gross Crest Width	10.7 m
Chute Width	varies 10.7 m to 16.0 m
Basin Width	16.0 m
Structure Length	119.0 m
Basin Elevation	943.0 m
Design Discharge (1:100 routed outflow)	78.6 m^3/s
Probable Maximum Discharge (0.5 PMF routed outflow)	228.9 m^3/s

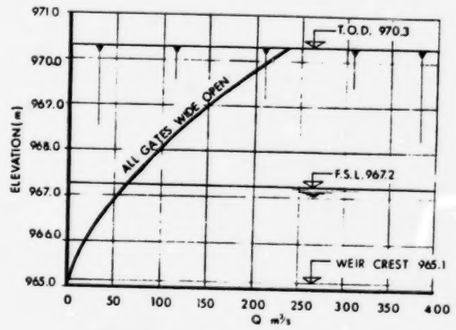
Irrigation Supply Conduit

Conduit Diameter	1.5 m
Conduit Length	158 m
Irrigation Supply Capacity Requirement	8.5 m^3/s
Gate Size	2 @ 2 m x 2 m
Intake Crest Elevation	949.0 m
Basin Elevation	944.0 m
Minimum Reservoir Elevation to Pass Irrigation Requirement	949.7 m

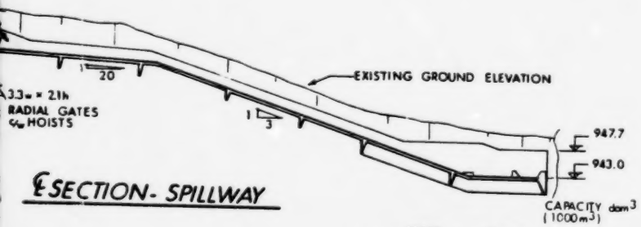




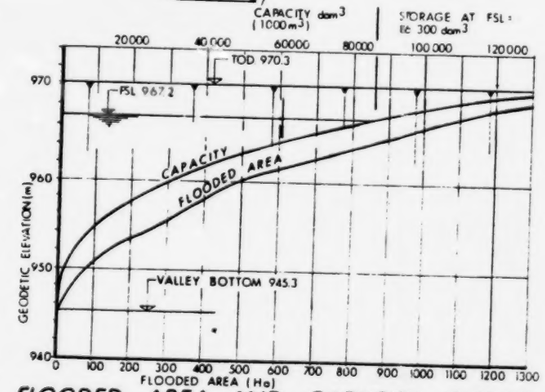
SPILLWAY PLAN



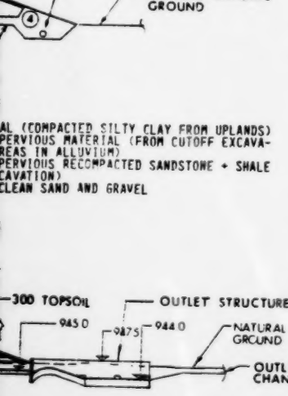
SPILLWAY RATING CURVE

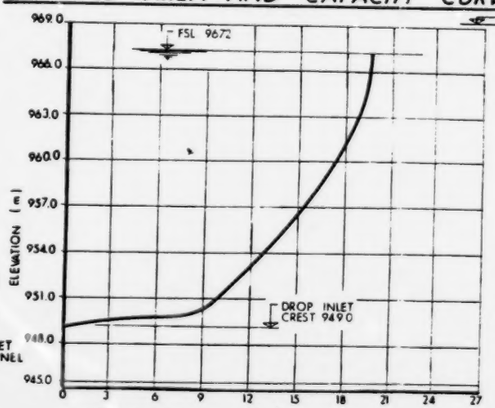


SECTION- SPILLWAY



FLOODED AREA AND CAPACITY CURVES





IRRIGATION SUPPLY CONDUIT RATING CURVE

REV	DATE	REVISION DESCRIPTION	ENG	APPR

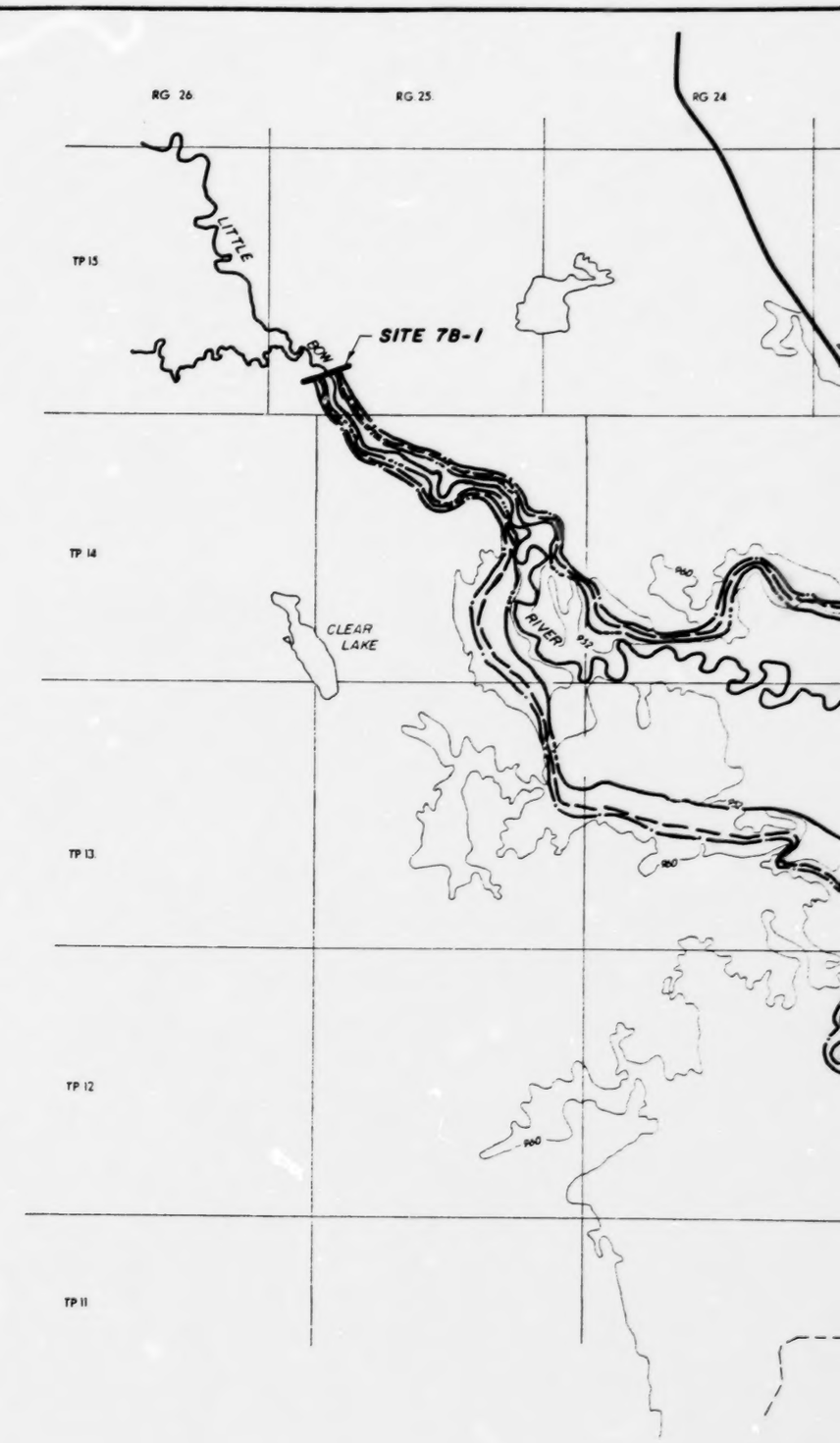
W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

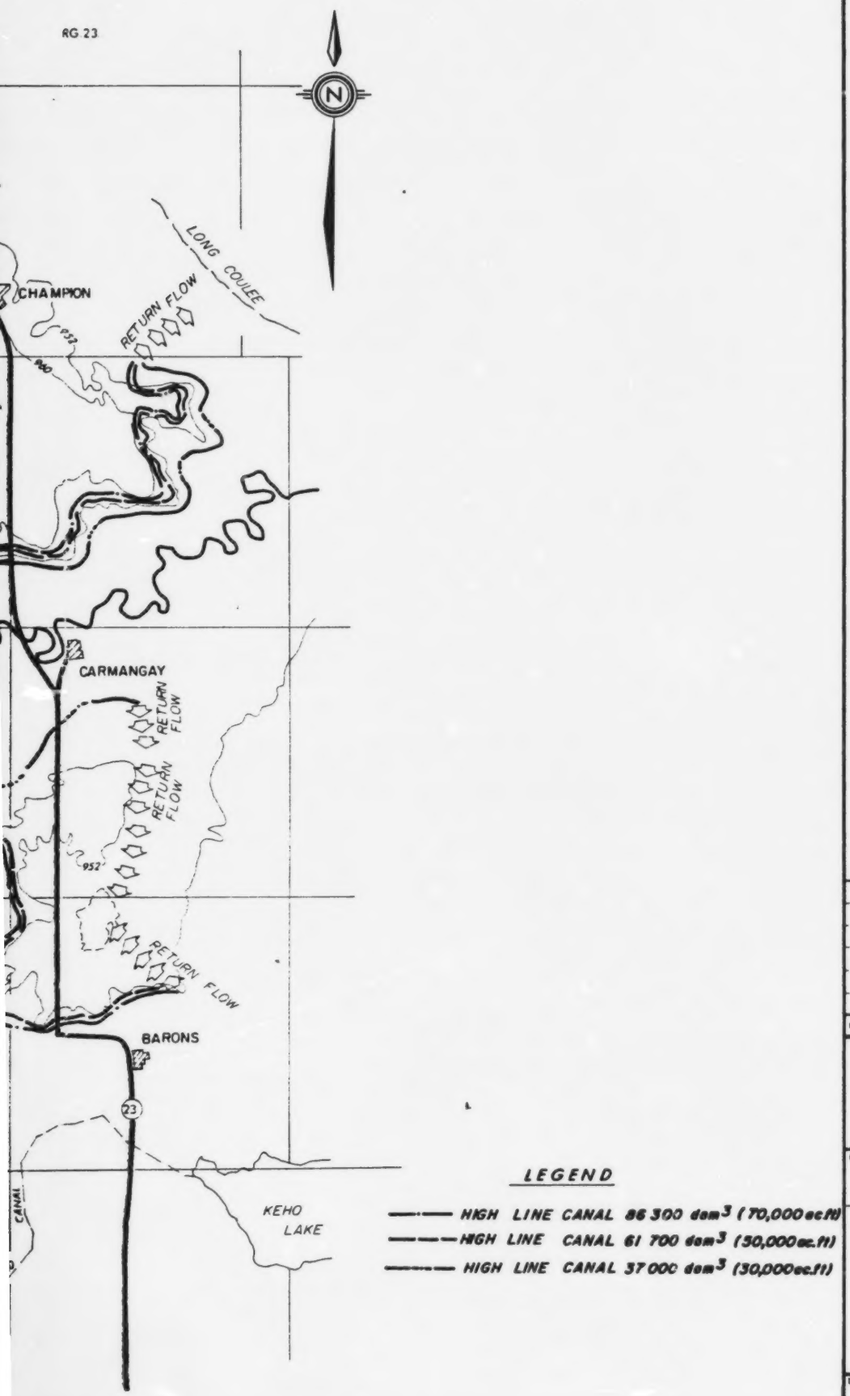
PROJECT
LITTLE BOW RIVER BASIN
STORAGE INVESTIGATION
CONFLUENCE SITE 7B-1
FEASIBILITY DESIGNS

86 300 dam³ STORAGE ALTERNATIVE
(70,000 ac.ft)

GENERAL LAYOUT

DESIGNED	CHECKED	DATE	JOB No	DRAWING No
P.B.				C3
DRAFTED	APPROVED	SCALE	SHEET No	
E.O.				





LEGEND

——— HIGH LINE CANAL 86 300 dam³ (70,000 ac.ft)

——— HIGH LINE CANAL 61 700 dam³ (50,000 ac.ft)

——— HIGH LINE CANAL 37 000 dam³ (30,000 ac.ft)

Notes:

REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT: LITTLE BOW RIVER BASIN
STORAGE INVESTIGATION
CONFLUENCE SITE

SITE 7B-1
POTENTIAL HIGH LINE
CANAL LOCATIONS

DESIGNED	CHECKED	DATE	JOB No	C4
DRAFTED P.G.	APPROVED	SCALE 1:50	SHEET No	

DATE: SEPT 17 85

LITTLE BOW RIVER BASIN
STORAGE INVESTIGATION
CONFLUENCE SITE 7B-1
FEASIBILITY DESIGN REPORT
September, 1985

APPENDIX D

RESERVOIR COSTS

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1.0 BASIS OF RESERVOIR COSTS

An estimate of the costs associated with land acquisition and reservoir damages at Site 7B-1 has been prepared, as detailed in subsequent sections of this appendix. Location plans of reservoir damages are shown on Figures D1 through D3.

1.1 Land Acquisition

The areas of land required for each of the three reservoirs were based on top-of-dam elevation to allow for reservoir surcharge during river flood stages and for possible future erosion of the reservoir abutments. The total reservoir areas were broken down into cultivation (including hay flats) and other land use (pasture, bush, river) as based on field inspections and air photo studies in conjunction with topographic plans of the reservoir. Unit land costs of $1980 per hectare (ha) of cultivation and $740 per hectare of other lands as used in a previous report (Ref 1) have been discussed with officials from the Municipal District of Foothills and are considered to be representative of current land values in this area. An additional allowance of 25% for serverance and acquisition cost has been included.

1.2 Reservoir Damages

Damages associated with the reservoirs included in this phase of the study include the following:

- roads and bridges
- power transmission lines
- telephone lines
- natural gas lines
- farm buildings
- water intake system

The costs associated with public utilities were estimated based on information supplied by Trans-Alta Utilities Corporation, Canadian Western Natural Gas Company Limited, and Alberta Government Telephones.

The costs associated with relocation of existing farm buildings effected by the reservoir were based generally on costs supplied by local moving companies in connection with a study of Keho Lake Enlargement (1980), updated to 1985.

The costs of raising existing roads and bridges were calculated separately for each road.

2.0 LITTLE BOW SITE 7B-1 - 37,000 DAM^3 (30,000 AC.FT.)

2.1 Land Acquisition

The total area of land affected by the reservoir to top-of-dam elevation is 776 ha, of which 176 ha are cultivated as tame hay flats and cropland. The costs of acquisition are estimated as follows:

Cultivation	176 ha @ $1980	=	$ 348,000
Other	600 ha @ $ 740	=	444,000
Severance (25%)		=	198,000
	TOTAL		$ 990,000

2.2 Reservoir Damages

2.2.1 Roads and Bridges

a) Secondary Highway (S.H.) 529

Secondary Highway 529 crosses the Site 7B-1 reservoir at the north boundary of Section 12-15-26-W4. Three alternate

routes for S.H. 529 were examined as described below:

i) Retain the current alignment of the road at the north boundary of Section 12-15-26-W4 and raise the existing road and bridge to form a causeway at a cost estimated at $2,800,000.

ii) Re-route the road three kilometres north to the south half of Section 26-15-26-W4. This would involve raising the existing road and bridge and upgrading from gravel to asphalt at an estimated cost of $1,400,000.

iii) Re-route the road three kilometres south as shown in Figure D1. This would require replacing two existing bridges and upgrading from gravel to asphalt at a cost estimated at $1,300,000. This is the most economical alternative and was incorporated in the reservoir costs assessment.

User costs were not included in this study for the six addition kilometres of travel associated with alternatives ii) and iii) for traffic estimated at 200 vehicles per day.

2.2.2 Farm Buildings

a) NE 5-15-25-W4

These buildings, although above the reservoir level could be affected by the spillway for Site 78-1. Relocation costs are estimated at $50,000.

b) NE 12-15-26-W4

The relocation of the occupied farmstead is estimated at $80,000.

c) **Hutterite Colony SE 13-15-26-W4**

The Little Bow River Hutterite Colony consists of a large number of buildings, some of which would have to be dismantled and re-constructed. The costs of dismantling and re-constructing the major buildings, plus relocation of the smaller structures, including re-connection of utilities are estimated at $1,000,000.

d) **SE 26-15-26-W4**

Relocation of this occupied farmstead is estimated to cost $80,000.

2.2.3 Utilities

a) **Powerlines**

A small single phase power line crosses near the upper end of the reservoir in the SE 26-15-26. Cost of the relocation is estimated at $70,000.

b) **Pipelines**

No pipelines were found to be affected by the reservoir with the exception of natural gas service lines to farm buildings which are to be relocated.

c) **Town of Vulcan Water Intake and Pumphouse**

The town of Vulcan water intake and pumphouse in the NE 35-15-26-W4 would have to be relocated. This would involve construction of a new intake line and pumphouse. Estimated cost of this work is $150,000.

d) Telephone Lines

One short section of telephone line in the SW 18-15-25 would have to be relocated at an estimated cost of $5,000.

TABLE D1
RESERVOIR DAMAGES
LITTLE BOW SITE 7B-1
37 000 dam^3 (30,000 ac.ft.)

Land Acquisition	990,000
Roads and Bridges	1,300,000
Farmstead Relocations	1,210,000
Utility Relocations	225,000
TOTAL	$3,725,000

3.0 LITTLE BOW SITE 7B-1 - 61 700 DAM^3 (50,000 AC.FT.)

3.1 Land Acquisition

The total area of lands affected by the reservoir to top-of-dam elevation is 1,139 ha, of which approximately 327 ha are cultivated as tame hay flats and cropland.

The cost of acquisition is estimated as follows:

Cultivation or Irrigation	327 ha. @ $1980	647,000
Other	812 ha. @ $740	601,000
Severance (25%)		312,000
	TOTAL	$1,560,000

3.2 Reservoir Damages

3.2.1 Roads and Bridges

Three alternative routes for Secondary Highway 529 were evaluated as per Sections 2.2.1 (i), (ii) and (iii). The results are as follows:

i) Estimated cost of $4,000,000
ii) Estimated cost of $1,700,000
iii) The most economical alternative at an estimated cost of $1,300,000

3.2.2 Farm Buildings

a) NE 5-15-25-W4

This occupied farmstead, described in Section 2.2.2(a) would be relocated at an estimated cost of $50,000.

b) **NE 12-15-26-W4**

The relocation of this occupied farmstead as described in Section 2.2.2(b) is estimated at $80,000.

c) **Hutterite Colony SE 13-15-26-W4**

The Little Bow River Hutterite Colony as described in Section 2.2.2.(c) of this report, would be flooded by the reservoir. The estimated cost of relocating is $1,000,000.

d) **SE 26-15-26-W4**

Relocation of this occupied farmstead is estimated to cost $80,000.

3.2.3 Utilities

a) **Powerlines**

The small single phase powerline as described in Section 2.2.3(a) would be relocated at an estimated cost of $70,000.

b) **Pipelines**

No pipelines were found to be affected by the reservoir, with the exception of farm services as described in Section 2.2.3 (b).

c) **Town of Vulcan Water Intake and Pumphouse**

Relocation of the water intake and pumphouse as described in Section 2.2.3(c) would be done at an estimated cost of $150,000.

d) **Telephone Lines**

One section of telephone line in the SW 18-15-25-W4 would have to be relocated at an estimated cost of $5,000.

TABLE D2
RESERVOIR DAMAGES
LITTLE BOW SITE 7B-1
61 700 dam^3 (50,000 ac.ft.)

Land Acquisition	$1,560,000
Roads and Bridges	1,300,000
Farmstead Relocations	1,210,000
Utility Relocations	225,000
TOTAL	$4,295,000

4.0 LITTLE BOW SITE 78-1 - 86 300 DAM^3 (70,000 AC.FT.)

4.1 Land Acquisition

The total area of lands affected by the reservoir to top-of-dam elevation is 1,783 ha, of which approximately 353 ha are cultivated as hay or cropland.

The cost of acquisition is estimated as follows:

Cultivation or Irrigation	353 ha.	@ $1980	699,000
Other	1430 ha.	@ $740	1,058,000
Severance (25%)			439,000
	TOTAL		$2,196,000

4.2 Reservoir Damages

4.2.1 Roads and Bridges

(a) Secondary Highway 529

Three alternative routes for Secondary Highway 529 were evaluated as per Sections 2.2.1 (i), (ii) and (iii). The results are as follows:

i) Estimated cost of $5,500,000
ii) Estimated cost of $2,300,000
iii) The most economical alternative at an estimated cost of $1,300,000

b) NW 9-16-26-W4

The existing timber farm bridge on the local road would be abandoned and local traffic re-routed over S.H. 533.

4.2.2 Farm Buildings

a) NE 5-15-25-W4

This occupied farmstead, described in Section 2.2.2(a) would be relocated at an estimated cost of $50,000.

b) NE 12-15-26-W4

The relocation of this occupied farmstead as described in Section 2.2.2(b) is estimated at $80,000.

c) Hutterite Colony SE 13-15-26-W4

Relocation of the Little Bow River Hutterite Colony is estimated to cost $1,000,000.

d) SE 26-15-26-W4

Relocation of this occupied farmstead is estimated to cost $80,000.

e) SE 8-16-26-W4

The relocation of this occupied farmstead is estimated to cost $100,000.

4.2.3 Utilities

a) Powerlines

The small single phase powerline as described in Section 2.2.3(a) would be relocated at an estimated cost of $70,000.

b) Pipelines

No pipelines were found to be affected by the reservoir, with the exception of farm services as described in Section 2.2.3 (b).

c) Town of Vulcan Water Intake and Pumphouse

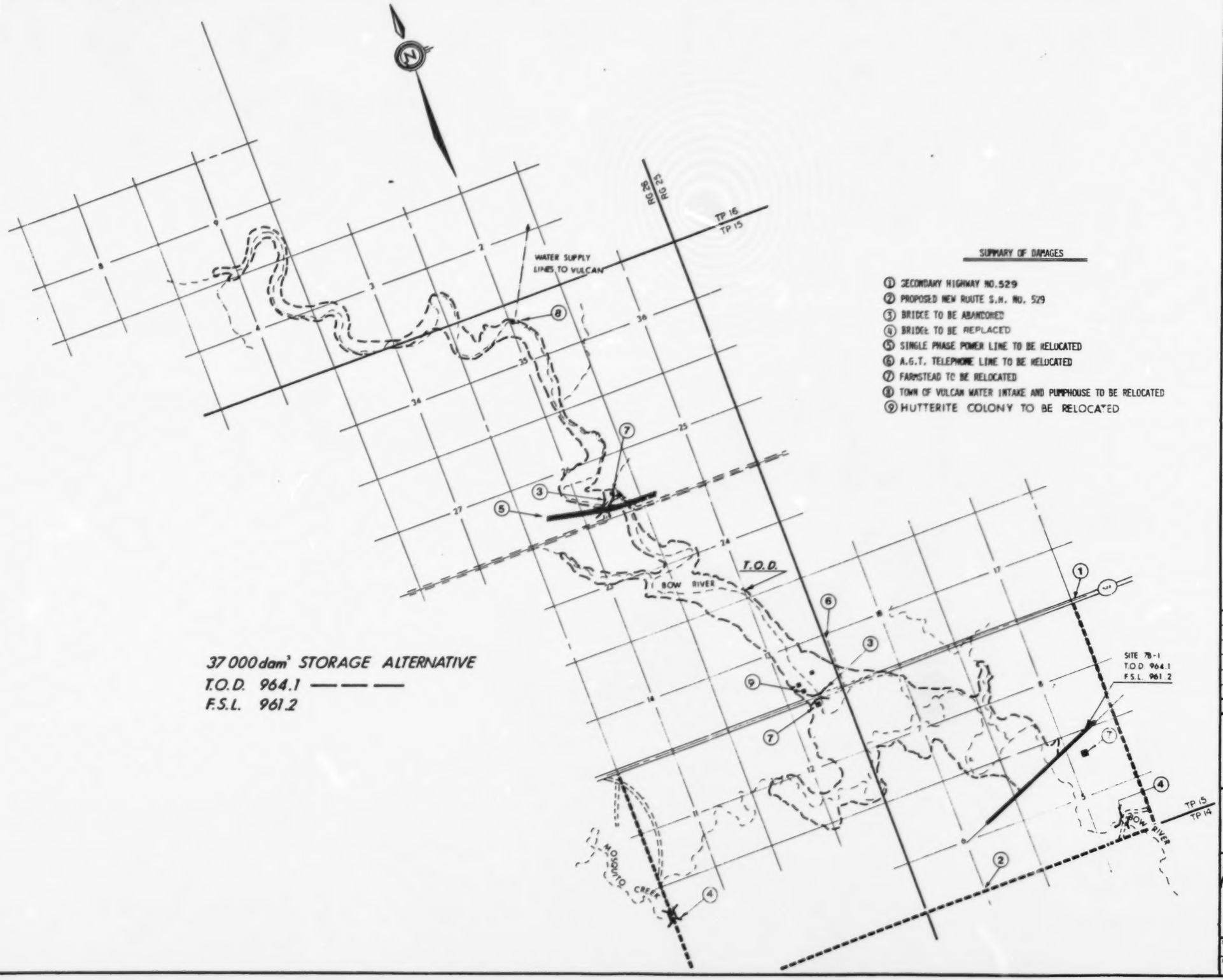
Relocation of the water intake and pumphouse as described in Section 2.2.3(c) would be done at an estimated cost of $150,000.

d) Telephone Lines

One section of telephone line in the SW 18-15-25-W4 would have to be relocated at an estimated cost of $5,000.

TABLE D3
RESERVOIR DAMAGES
LITTLE BOW SITE 7B-1
86 300 dam^3 (70,000 ac.ft.)

Land Acquisition	$2,196,000
Roads and Bridges	1,300,000
Farmstead Relocations	1,310,000
Utility Relocations	225,000
TOTAL	$5,031,000



37 000 dam³ STORAGE ALTERNATIVE
T.O.D. 964.1 — — —
F.S.L. 961.2

SUMMARY OF DAMAGES

① SECONDARY HIGHWAY NO.529
② PROPOSED NEW ROUTE S.H. NO. 529
③ BRIDGE TO BE ABANDONED
④ BRIDGE TO BE REPLACED
⑤ SINGLE PHASE POWER LINE TO BE RELOCATED
⑥ A.G.T. TELEPHONE LINE TO BE RELOCATED
⑦ FARMSTEAD TO BE RELOCATED
⑧ TOWN OF VULCAN WATER INTAKE AND PUMPHOUSE TO BE RELOCATED
⑨ HUTTERITE COLONY TO BE RELOCATED

Notes:

REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT
LITTLE BOW RIVER BASIN
STORAGE INVESTIGATION
CONFLUENCE SITE 7B-1
FEASIBILITY DESIGNS

37 000 dam³ STORAGE ALTERNATIVE (30,000 ac.ft)
RESERVOIR DAMAGES SUMMARY PLAN

DESIGNED	CHECKED	DATE	JOB No	DRAWING No
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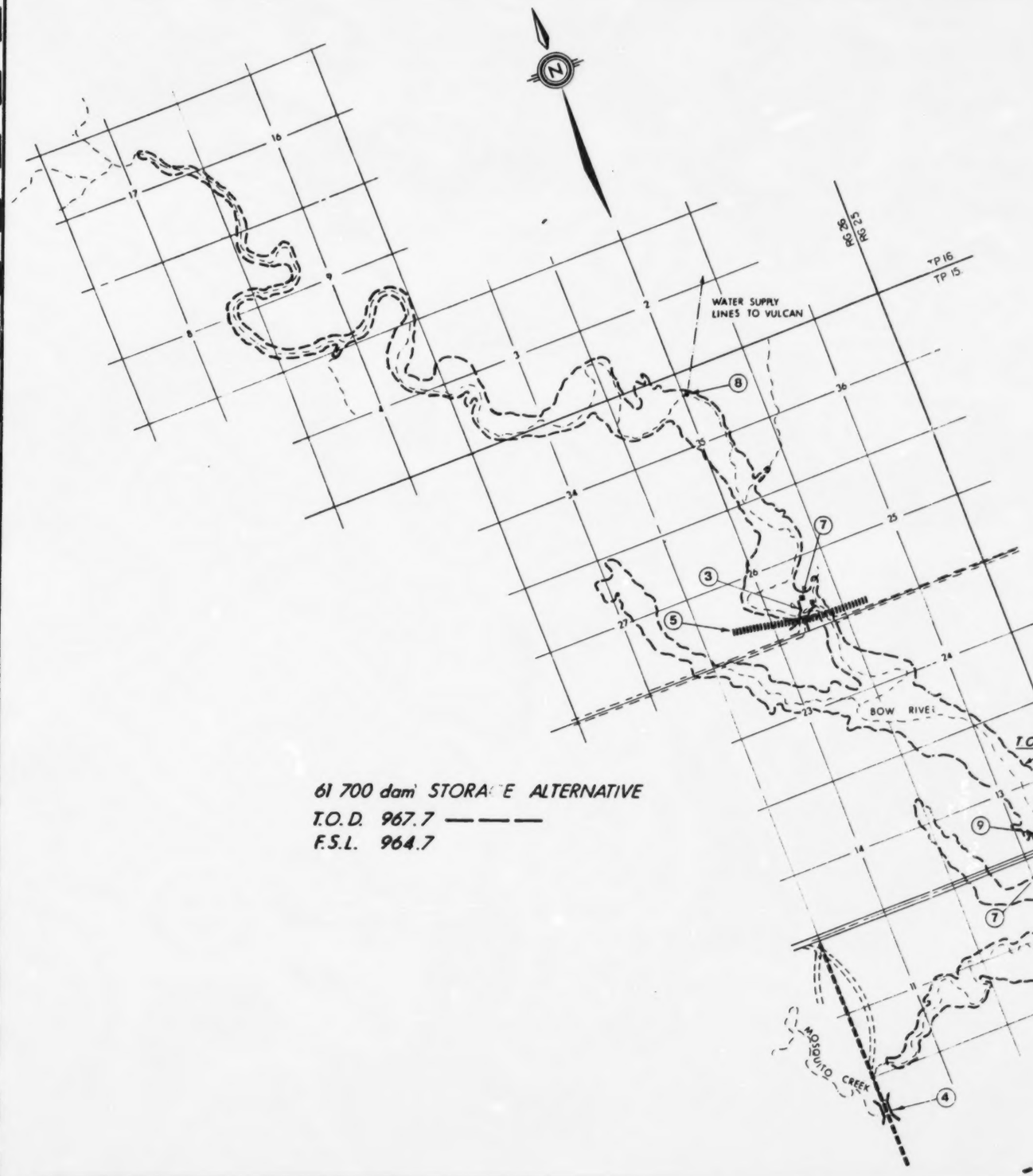

SUMMARY OF DAMAGES

① SECONDARY HIGHWAY NO529
② PROPOSED NEW ROUTE S.H. NO. 529
③ BRIDGE TO BE ABANDONED
④ BRIDGE TO BE REPLACED
⑤ SINGLE PHASE POWER LINE TO BE RELOCATED
⑥ A.G.T. TELEPHONE LINE TO BE RELOCATED
⑦ FARMSTEAD TO BE RELOCATED
⑧ TOWN OF VULCAN WATER INTAKE AND PUMPHOUSE TO BE RELOCATED
⑨ HUTTERITE COLONY TO BE RELOCATED

Notes:

REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY ALBERTA

PROJECT
LITTLE BOW RIVER BASIN
STORAGE INVESTIGATION
CONFLUENCE SITE 7B-1
FEASIBILITY DESIGNS

61 700 dam³ STORAGE ALTERNATIVE
(50,000 ac-ft)
RESERVOIR DAMAGES
SUMMARY PLAN

DESIGNED	CHECKED	DATE	JOB No	DRAWING No
P.B		85/10/01		D2
DRAFTED	APPROVED	SCALE	SHEET No	
E.C				

86 300 dam³ STORAGE ALTERNATIVE
T.O.D. 970.3 — — —
F.S.L. 967.2

SUMMARY OF DAMAGES

① SECONDARY HIGHWAY NO.529
② PROPOSED NEW ROUTE S.H. NO. 529
③ BRIDGE TO BE ABANDONED
④ BRIDGE TO BE REPLACED
⑤ SINGLE PHASE POWER LINE TO BE RELOCATED
⑥ A.G.T. TELEPHONE LINE TO BE RELOCATED
⑦ FARMSTEAD TO BE RELOCATED
⑧ TOWN OF VULCAN WATER INTAKE AND PUMPHOUSE TO BE RELOCATED
⑨ HUTTERITE COLONY TO BE RELOCATED

Notes:

REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT
LITTLE BOW RIVER BASIN
STORAGE INVESTIGATION
CONFLUENCE SITE 7B-1
FEASIBILITY DESIGNS

86 000 dam³ STORAGE ALTERNATIVE (70,000 ac.ft)
RESERVOIR DAMAGES
SUMMARY PLAN

DESIGNED	CHECKED	DATE	JOB No	DRAWING No
P.B.		Sept 27 '84		D3
DRAFTED	APPROVED	SCALE	SHEET No	
E.O.				